CHAPTER 3
TEST PROGRAMME ON CONNECTIONS BETWEEN PRECAST CONCRETE ELEMENTS

3.1 INTRODUCTION

The information regarding different connection details to be investigated in the experimental programme for this project was gathered from several consulting engineering firms. This information was used to identify the preferred structural systems in New Zealand for moment resisting frames incorporating precast concrete elements, and to obtain specific details on the arrangements used for connecting the precast elements together. It was found that for buildings between 9 and 21 storeys in height the general trend is to allocate the whole of the earthquake resistance to stiff perimeter frames. Typically, the clear span to overall depth ratio for beams ranges from 3 to 6 while for columns in the upper floors of the building the clear height to overall depth ratio varies between 2.5 and 4. Hence, the emphasis in this experimental programme is on precast concrete components representative of perimeter frames.

3.2 TEST UNITS

3.2.1 Design Considerations

The test programme involved the testing of six subassemblages and the repair work and retesting of one of them. The connection details between the precast concrete elements of the subassemblages were typical of those already described in Section 1.4.1. Fig. 3.1 illustrates the origin of the subassemblages tested. The first four units, Units 1 to 4, were H-shaped subassemblages representing components of Systems 2 or 3 in the lower storeys of a perimeter frame. There, the flexural capacity of the beams may be limited by the maximum level of shear in the potential plastic hinge regions permitted by the Concrete Design Code [NZS 3101 (1982)] if diagonal reinforcement is to be avoided. Units 5 and 6 were cruciform-shaped subassemblages and represented components of Systems 1 and 2 in the upper floors of a perimeter frame where the absence of high axial compression loads in the columns makes the most unfavourable condition for the beam-column joint region.

One feature of perimeter frames is that gravity loading does not have a significant influence on the bending moment diagram of the beams when combined with the full seismic action and therefore both negative and positive moments are of similar magnitude. Also, the point of contraflexure remains near midspan. Fig. 3.2 displays a typical bending moment diagram of a beam of a perimeter frame. Often in practice the designer uses a small moment redistribution to permit equal amounts of top and bottom longitudinal reinforcing steel to be placed in the beams.
Fig. 3.1 - Origin of Test Units.

Fig. 3.2 - Typical Bending Moment Diagram of a Beam of a Perimeter Frame.
Table 3.1
Theoretical Stiffness and Drift of Test Units

<table>
<thead>
<tr>
<th>Unit</th>
<th>Dependable Lateral Load Strength $H_D$</th>
<th>Theoretical Stiffness $K_{th}$</th>
<th>Theoretical Drift at $H_D$</th>
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<tr>
<td></td>
<td>kN</td>
<td>MN/m</td>
<td>%</td>
</tr>
<tr>
<td>1</td>
<td>350.2</td>
<td>124.0</td>
<td>0.13</td>
</tr>
<tr>
<td>2</td>
<td>350.2</td>
<td>124.0</td>
<td>0.13</td>
</tr>
<tr>
<td>3</td>
<td>346.7</td>
<td>124.0</td>
<td>0.12</td>
</tr>
<tr>
<td>4</td>
<td>535.0</td>
<td>133.8</td>
<td>0.18</td>
</tr>
<tr>
<td>5</td>
<td>254.9</td>
<td>40.6</td>
<td>0.28</td>
</tr>
<tr>
<td>6</td>
<td>252.6</td>
<td>40.6</td>
<td>0.28</td>
</tr>
</tbody>
</table>

(1) Based on the specified properties of the materials and assuming a rectangular stress block for the compressed concrete, a bilinear stress-strain relation for the steel and a strength reduction factor $\phi = 0.9$.

(2) Based on one-half of the gross section values for the beams and columns and accounting for flexural and shear deformations only.

The design of the units with regard to interstorey drift complied with the Loadings Code [NZS 4203 (1984)]. This code stipulates that the theoretical interstorey drift for ductile reinforced concrete frames in which all partitions are separated, when acting elastically when subjected to the design seismic loading, shall not exceed 0.33% of the story height. The theoretical interstorey drifts, presented in Table 3.1, were calculated based on the dimensions of the test units following an elastic analysis. To account for cracking in the elements only one half of the gross section properties for flexure and shear of beam and columns was considered. Other sources of flexibility were not included in order to follow normal office procedures. The dependable lateral load strength was used to establish this theoretical interstorey drift.

3.2.2 Description of the Test Units

Figs. 3.3 to 3.8 show complete reinforcing details of all test units. The dimension of the test specimens were near full scale. The effect of a proprietary precast floor system was outside of the scope of this project and therefore the prototypes were slabless.

3.2.2.1 Units 1, 2 and 3

Units 1, 2 and 3 were conventionally reinforced precast concrete units connected at midspan by short lap splices. The two main points of interest to be investigated in the tests on these units were:
Fig. 3.3 - Reinforcing Details of Unit 1.

Fig. 3.4 - Reinforcing Details of Unit 2.
Fig. 3.5 - Reinforcing Details of Unit 3.

Fig. 3.6 - Reinforcing Details of Unit 4.
Fig. 3.7 - Reinforcing Details of Unit 5.

Fig. 3.8 - Reinforcing Details of Unit 6.
a) The behaviour of the connection detail at midspan.

b) The proximity of the lap splice to the critical region in the beams at the column faces.

One difficulty encountered connecting short precast concrete beams in accordance with the Concrete Design Code prior to the recent Amendment No.1 was that when plastic hinges are expected to develop in the beams at the column faces no part of the splice of the longitudinal reinforcement was permitted to occur within a distance of 2d from the column face where d is the effective depth of the beam. Thus, the clear span of the beam had to be at least 4d + l, where l is the splice length. To satisfy this requirement in practice it was necessary to reduce as far as possible the length of the splice and/or to reduce the distance the splice is permitted to commence from the column face without adversely affecting the seismic response of the structure. The alternative of reducing the beam depth to increase the aspect ratio was not often viable since the size of these elements is largely dictated by stiffness needed to satisfy the interstorey drift requirements. A clear span to overall depth ratio of 3 was chosen for the beams tested in Units 1, 2 and 3 to represent the shortest concrete beam normally encountered in practice in such frames. The units were designed to develop plastic hinges in the beams at the column faces. Thus, using capacity design, the columns and beam-column joints were made overstrong. The midspan connections were tested under the maximum shear permitted by the Concrete Design Code when symmetrically reinforced beams are expected to develop plastic hinges and diagonal reinforcing steel in these regions is to be avoided. In terms of nominal shear stresses this limitation is equal to \( v^* = V^*/bd = 0.30\sqrt{\frac{E}{F}} \) evaluated at a flexural overstrength of 1.25, where \( V^* \) is the beam shear at overstrength, b is the width of the beam and d is the effective depth of the beam. Hence the longitudinal reinforcement ratio of the beams based on Grade 300 steel was \( \rho = 0.81\% \) and a nominal concrete compressive strength of 30MPa was specified.

The midspan connection detail of Unit 1, shown in Fig. 3.3, consisted of overlapping 180° hooks commencing at 1.46d from the critical region at the column face. Four D28 transverse rods were placed in contact with the inside of the bends of the hooks. D16 stirrups surrounded the connection in the cast in place concrete joint and was arbitrarily chosen following an existing detail. This reinforcement was, according to the nominal properties of the steel, able of transferring 115% of the beam shear at overstrength. Transverse steel elsewhere in the beam was controlled by the shear demand in the plastic hinge regions. All the longitudinal steel in the beam was spliced using this detail involving overlapping 180° hooks at midspan and kinking of the longitudinal bars was necessary to offset them in the connection. An additional stirrup was placed at the kink.

Unit 2 was connected at midspan using double 90° hooked "drop in" bars as shown in Fig. 3.4. The connection detail commenced at a distance 1.23d from the column face. As in Unit 1, transverse rods were tied in contact with the inside of the hooks of the lapped bars. Two-thirds of the bars were spliced using this detail at midspan and one-third of the beam longitudinal bars were anchored within the precast concrete members. This curtailment was deliberately made to observe the effects of higher stresses in the longitudinal steel, up to yielding, in the connection region. The transverse reinforcement in the connection region was capable of transferring 80% of the beam shear at
overstrength. Elsewhere in the beam the transverse reinforcement provided was governed by the shear requirements in the plastic hinge regions.

Fig. 3.5 depicts the complete reinforcing details of Unit 3. The midspan connection detail consisted of non-contact lap splices commencing at 1.27d from the column faces. The lap length of 23dₖ used accounted for the direction of concreting or top bar effect and the amount of transverse steel surrounding the lap. As for Unit 2 only two thirds of the longitudinal bars were spliced at midspan. The transverse reinforcing steel in the plastic hinge regions was controlled by the anti-buckling requirements. The transverse reinforcement in the connection region was governed by the need to transfer the forces between the splices by a 45° truss as suggested by Paulay (1982).

3.2.2.2 Unit 4

Unit 4, shown in Fig. 3.6, had a diagonally reinforced midspan connection scaled from a real component. This arrangement has been used when the nominal shear stress at overstrength, \( \nu^o \), in the potential plastic hinge regions has exceeded 0.30\( \sqrt{\nu} \). For nominal shear stresses \( \nu^o = \nu^o/bd > 0.30\sqrt{\nu} \), the Concrete Design Code requires the use of diagonal reinforcing steel to resist the shear force in that region, so as to avoid loss of energy dissipation and strength degradation caused by an early sliding shear failure. This Code requirement makes it difficult to detail reinforcement for short precast concrete beams falling in this category. However, the beam longitudinal reinforcement can be detailed to preclude the formation of plastic hinges at the column faces and hence to concentrate all plasticity into the beam midspan region where the diagonal reinforcing steel can be easily fitted. This solution has been presented by Buchanan (1979) and tested by Bull (1978) for cast in place construction. It can be modified for precast concrete construction by welding the diagonal bars to steel plates which are bolted at midspan. This arrangement also has the advantage that the beam-column joint region can be designed to be less congested because the Concrete Design Code eases the transverse joint reinforcement requirements if the adjacent portions of the beam remain in the elastic range.

The dimensions of the real diagonally reinforced component were adjusted within the dimensions of Units 1 to 3. The design used the simple truss model illustrated in Fig. 3.9. At the bend

![Fig. 3.9 - Simple Truss Model Assumed for the Design of Unit 4.](image-url)
of the diagonal bars, 30x10 mm vertical steel straps were provided to transfer one half of the total shear force with an overstrength factor of 1.25. These straps were cut from a flat bar, formed to a "C" shape and joined by butt welding the ends. Four extra D28 bars were provided in the strong regions of the beam next to the column faces to preclude the formation of plastic hinges in this region. These bars were anchored with 90° hooks adjacent to the bend in the diagonal bars. The D24 diagonal bars were fillet welded over a length equal to 4dₜ to 16mm thick mild steel plates at the midspan of the beam. The D24 and D28 were bent using a 140 mm diameter mandril, equivalent to a diameter of bend of 5.8dₜ and 5dₜ, respectively. A 20mm thick sandwich steel plate was used for interconnecting the steel plates of the precast components at midspan. The holes in this plate were 2mm oversized on one side and slotted on the other side to give ample construction tolerances. M22/TF high strength friction grip bolts passing through the 16mm and the 20mm thick steel plates were used to interconnect the precast concrete units. The bolts were designed to transfer by friction the total shear force at overstrength plus the small moment arising from the eccentricity between the line of action of the bolts and the point of contraflexure at midspan. A coefficient of friction of 0.32 was assumed in the design.

Unit 4 was repaired after testing and retested. Repair details are presented in Chapter 5 together with test results.

3.2.2.3 Units 5 and 6

The cruciform shaped units tested, Units 5 and 6, had beams with an equivalent clear span to overall depth ratio of 4.58. Both units had identical dimensions and were designed to develop plastic hinges in the beams at the column faces.

Unit 5, shown in Fig. 3.7, had a beam-column joint with a cast in place joint core. The bottom bars of the beams were anchored inside the core of the beam-column joint, which contravened the requirements of the Concrete Design Code [NZS 3101 (1982)], which recommends that longitudinal beam bars that are intended to be terminated at an interior column be passed right through the core of the column and terminated with a standard hook immediately outside the far side of the hoops enclosing the beam-column joint.

Some of the potential problems to be investigated in this system were:

a) The effect of the hooked anchorage of the bottom bars of the beams inside the beam-column joint core on the overall behaviour of the joint.

b) The influence of the cold joint between the end of the precast beam and the column face when the plastic hinge develops in that region.

The longitudinal steel reinforcement of the beams, based on Grade 300 steel, was \( \rho = 0.93\% \) and \( \rho' = 0.95\% \). D24 bars were selected as top reinforcing steel since the code limits the maximum bar diameter to \( h/25 \). D28 bars were chosen as bottom reinforcing steel to reduce congestion and because it was the largest bar diameter that could be anchored in the joint core. Section C-C in Fig. 3.7 shows the positioning of the hooked bars inside the beam-column joint region. The outer D28 bars were
kinked inside the joint core and an additional R10 tie was detailed for the out of plane forces induced in the kink. The inner D28 bars were 20mm offset from the beam vertical centreline. The end of the precast concrete beams were seated on the 30mm of concrete cover of the lower column. If that cover concrete remained intact, it was calculated that the beam shear at overstrength could be transmitted by bearing between the precast member and the concrete cover of the column below, based on a maximum bearing stress at an overstrength of $0.85f'$. The transverse reinforcement in the potential plastic hinges of the beams was governed by the antibuckling requirements. The hoops in the beam-column joint were designed to take all the horizontal shear in this region.

Unit 6, depicted in Fig. 3.8, had a precast concrete element which passed through the beam-column joint region. The precast concrete member had vertical ducts in the joint core region to allow the protruding column bars to pass through. The horizontal construction joint between the precast beam and column and the vertical ducts was grouted in one operation. Potential problems to be investigated in this system were:

   a) The effectiveness of the grout around the longitudinal column bars, which needs to provide adequate bond and to permit adequate transfer of the transverse forces from the joint hoops to the vertical column bars.

   b) The performance of the grouted horizontal construction joints between the precast beam and the columns.

The beams of Unit 6 had 4-D24 bars as top and bottom reinforcing steel; that is $\rho = \rho' = 0.93\%$. The vertical ducts in the joint core region of the precast beams were formed using corrugated 70mm overall diameter steel tubes made from 0.4mm thick by 36mm wide galvanized steel sheets. The amount of transverse steel in the beams and in the beam-column joint region was identical to that used in Unit 5. However, the arrangement of the column reinforcement in Unit 6 was slightly changed due to the different positioning of the beam bars and the larger spaces required by the corrugated vertical ducting.

### 3.3 Loading Frames

Two different loading frames were designed for the experimental programme. They are shown in Fig. 3.10. Appendix B contains construction details of both frames.

The loading frame LF1 designed for the testing of the H-shaped Units 1, 2, 3 and 4 is shown in Fig. 3.10 (a). The main consideration in the design of this frame was to be able to apply equal displacements at each column to induce a bending moment in the beam similar to that normally encountered in the beam of a perimeter frame shown in Fig. 3.2. That is, the interstorey drift applied to each column of the units was identical. Two 1000kN independent hydraulic double-acting actuators were used to apply the predetermined cyclic test sequence. The hydraulic actuators reacted against the testing floor and induced some axial tension and compression loads on the columns, which had to be
Fig. 3.10 - Loading Frames Used in the Test Programme.
distributed from the base of the column to the strong floor. In addition, a sliding mechanism was provided at the bottom support of one column to allow for the elongation of the beams during the test.

Friction forces arising from the sliding mechanism caused a redistribution of the applied forces. The effects of the frictional force on the bending moment diagram of the beam can be seen in Fig. 3.11. The frictional force could not be directly monitored but a prediction of it was possible based on equilibrium considerations and the ratio of the measured loads at each jack. Fig. 3.12 depicts the predicted coefficient of friction from the measured loads on the hydraulic actuators. During the test of Unit 1 a moderate level of friction forces developed during the test. The sliding mechanism was modified with Permaglide T20 teflon strips to reduce the friction force in subsequent tests. Following the instructions from the manufacturer, the teflon strips were run in against the sliding steel block to overcome the initial friction of the virgin material. The measured coefficient of friction varied with the level of axial load being 3% at a pressure of 80MPa and larger at smaller pressures.

The loading frame LF2, designed for the test of Units 5 and 6, had only one double acting 1000kN hydraulic actuator as seen in Fig. 3.10 (b). The 567x450mm reinforced concrete column of Unit 4 was used as part of the frame to provide sufficient transverse and torsional stiffness. It was necessary to strain gauge the beam end supports and form a load sensitive device to determine the distribution of the applied lateral forces. These supports were inclined at 12.3° to the perpendicular planes of the beam to provide torsional and lateral restraint to the beams.

3.4 CONSTRUCTION OF THE SPECIMENS

3.4.1 Formwork

The formwork for the specimens was manufactured using 19mm plywood sheets. The moulds were stiffened with timber battens, steel angles and brackets at distances of no more than 400mm apart to minimize any bowing during the concreting of the specimens. In addition, steel rods placed inside a plastic hose passed through the columns of Units 5 and 6. The moulds were painted with undercoat and for each pour they were repainted and oiled. All edges were sealed with parcel tape and silicone to avoid bleeding of the water and segregation of fines from the fresh concrete. A commercial retarder, RUGASOL, was spread to the plywood mould on the vertical construction joints of Units 1 to 5. In addition, the retarder was also spread on to the base of the mould of the beam of Unit 6 in the horizontal construction joint with the lower column.

3.4.2 Reinforcing Cages

Most of the bars selected for instrumentation had either tack welded studs or strain gauges attached before the reinforcing cages were made. Only in one case, the repair of Unit 4, were strain gauges placed on bars when the reinforcement was in the test unit. In another case, Unit 5, tack weld of studs in the beam reinforcing at the beam-column joint region was carried out with the bars in its final position. The procedure for this operation will be discussed later in this chapter.
(a) Combined Gravity and Lateral Loading

(b) Friction Forces

Fig. 3.11 - Effect of Friction Forces Arising at the Sliding Mechanism in Loading Frame LF1.

Fig. 3.12 - Coefficient of Friction Estimated to Have Developed in the Sliding Mechanism.
Stirrups, hoops and ties were all cut and bent by a local firm. Despite the accuracy required, it was not possible to shape this reinforcing to within ±5mm. Main bars were all cut and bent in the concrete laboratory where closer tolerances could be attained. It was decided that all extension tails of the 90° anchorage hooks of longitudinal bars would be equal to 8dₜ instead of the standard 12dₜ. Stirrups and hoops in the test specimens were tied to the longitudinal bars within an accuracy of one-half of their own diameter.

The construction of the reinforcing cages for Unit 4 involved several extra steps. It was decided to incorporate the bent D24 bars in a strain aged condition. The strain ageing at the bent of the D24 diagonal reinforcement was accelerated by placing the bent bars in an oven at 97°C for 12 hours, which is equivalent to a natural strain ageing of 23 months at ambient temperature of 15°C [Erasmus (1987), Hundy (1954)]. This procedure was carried out because there has been some concern regarding strain age embrittlement of bent bars at the critical region of a member designed to respond inelastically during an earthquake [Erasmus (1978, 1981), Yap (1986)]. A quarter of the bars were not artificially strain aged as they had already been instrumented with strain gauges and the glue used for bonding them was susceptible to the oven temperature. The diagonal bars were fixed to a jig in contact with the 16mm end plate and fillet welded. A class S type of welding, in accordance with the Arc Welding Standard for Reinforcing Bars [NZS 4702 (1982)], was specified for the job. MIG welding was chosen because it had the advantage of no leaving slag and enabled uninterrupted welding runs to be carried out. Three probes were made to assess the quality and strength of the weld. One probe was sectioned to examine the penetration and quality of the weld. Fig. 3.13 shows that there is an underfill between the reinforcing bar and the steel plate that can not be relied on for strength purposes. Because of the critical consequences of a weld failure in this region, it was specified that the weld should commence from the bar midheight and with a slope to the horizontal not larger than 60°. Tensile tests on the remaining two trial samples showed fracture of the reinforcing bars at a section away from the weld region. That is, the weld did not affect the ultimate strength nor the ultimate strain characteristics. Details of the welding procedure are presented in Fig. 3.14.

Fig. 3.13 - Close up of a Probe to Examine the Quality of MIG Welding of a D24 Bar to a Steel Plate in Unit 4.
APPENDIX A  NZS 4702:1982
WELDING PROCEDURE QUALIFICATION DETAIL SHEET
FOR GRADE 275 REINFORCING STEEL

Contract .................................. Welding procedure No. ...........
Weld class ................................ S .........................

1. Bar diameter (plain/deformed) ...... Ø24 .........................
2. Joint preparation ........................
3. Height of reinforcement above main body of bar Full bar contact ...
4. Welding process MIG .....................
5. Filler metal specification and classification Mild Steel MIG Welding Wire 1.2mm Diameter ..................................
6. Shielding gas: Composition ARGOshield20 ............
Flow rate 10 l/min ..................
7. Power source characteristics 200 amps, 30 v dc, Gun to + (a.c. and open circuit voltage or d.c. and polarity)
8. Position of welding Horizontal right to left (including direction of vertical welding)
9. Root treatment (back gouging etc.) N/A .....................
10. Preheating and interrun temperature (250 °C max.) NQ ..........
11. Run details Fillet

<table>
<thead>
<tr>
<th>Run no.</th>
<th>Electrode diameter</th>
<th>Welding current</th>
<th>Welding* voltage</th>
<th>Joint detail sketch – include run sequence</th>
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<tr>
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<td>1.2</td>
<td>200</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>1.2</td>
<td>200</td>
<td>30</td>
<td></td>
</tr>
</tbody>
</table>

* For gas-shielded processes only.

This procedure may vary due to fabrication sequence, fit-up, bar diameter, runs and so on, within the changes permitted in Appendix B of NZS 4702.

Welder’s name ........................................ R. Allen Approved by ..................
................................................ Date ..................

Fabricator ............................................

Fig. 3.14 - Specifications of Welding Procedure.
Next, the reinforcing cages of Unit 4 were assembled (see Fig. 3.15). Care was exerted to ensure full contact between the outer D24 diagonal bars and the 30x10mm straps in the bend region. The straps were C-shaped and were connected by being butt welded at the top and bottom sides of the beam.

The cages of all units were placed in the formwork including additional 10mm diameter rods required to hold part of the instrumentation during the test. A length of wire spring and a plastic hose was fitted to the instrumentation rods and welded studs to create an annular void in the concrete cover. Special care was taken with the alignment of the steel sleeves at the column and beam ends since small errors would introduce large misalignments of the units in the loading frame. The sleeves were cut and milled at right angles, the positioned in the formwork using a high precision bubble level and locked in position with interior plywood disks. In addition, the sleeves of the columns of Units 1 to 4 were fixed to the sides of the mould with steel straps. A similar procedure was carried out to lock the corrugated ducts in Unit 6. Finally, lifting hooks were tied to the reinforcing cages of each unit. Fig. 3.15 shows several reinforcing cages prior concreting.

3.4.3 Concreting of the Precast Units

The concreting of Units 1 to 4 was carried out with the frame units in the horizontal plane. On the other hand, Units 5 and 6 were concreted in vertical position. The concrete was placed using a hopper and was mechanically vibrated. The exposed surface of the horizontal top test Units 1 to 4 and 6 was floated a few hours after casting of the concrete to obtain a smooth surface. The horizontal top surface of the beam of Unit 5 was left untouched but the construction joints of the columns of Units 5 and 6 were scrubbed 24 hours after being cast.

All units were cured for seven days with damp hessian fabrics and covered with plastic sheets. Immediately after removing the plywood mould, the paste affected by the retarder in the construction joints of the units was wire brushed. The cold joint showed a rather smooth face, with a total amplitude of about 2mm. It was also found that the coarse aggregate tended to lie flat against the walls of the formwork as shown in Fig. 3.16. No further scrubbing was done because it was considered that this smooth surface could be near the "worst" situation found in practice.

3.4.4 Preparation of the Connection

The connection of all units was assembled with the precast components in the loading frame. The stirrups in the midspan region of Units 1, 2 and 3 were placed temporarily in the one precast unit in the loading frame. The second precast unit was mounted and the midspan connection detail was prepared by sliding the stirrups along. All the stirrups in this region were 5mm oversized to ease the in situ work. Figs. 3.17 to 3.19 illustrate the reinforcement details in the midspan connection regions of these units.

The connection of the precast elements of Unit 4 using the 20mm thick sandwich plate was carried out with both precast sub-components placed in the loading frame. This plate showed some
(a) Unit 2 before Casting of the Concrete

(b) Unit 4 before Casting of the Concrete

(c) Unit 6 being Cast

Fig. 3.15 - Reinforcing Cages prior Concreting.
Fig. 3.16 - View of Vertical Cold Joint in a Beam of Unit 5.

(a) Elevation  (b) Plan View

Fig. 3.17 - Midspan Connection Detail of Unit 1.

(a) Elevation  (b) Plan View

Fig. 3.18 - Midspan Connection Detail of Unit 2.
incipient rust because it had been wetted and dried for about two weeks. High friction grip bolts with coronet load indicator washers were inserted between the plates and manually pre-tightened using a spanner. The final tightening was carried out in a staggering pattern using an impact wrench until the protrusions on the coronet load indicator washers were nearly flattened. When the cage around the connecting plates was completed, the formwork was set-up and the concrete was cast. Fig. 3.20 illustrates the midspan connection of Unit 4.

For Unit 5, the precast concrete beams were seated on the cover of the column below, which had already been pinned on the bottom bracket of the loading frame. The beams were seated in contact with the concrete of the column without any mortar or similar material between. The next step consisted of sliding the joint reinforcement and then top beam reinforcement. Fig. 3.21 shows the arrangement of the reinforcement in the beam-column joint region prior concreting.
The labour involved in the preparation of the joint of Unit 6 was minimal since the reinforcement was all included in the precast members. The precast concrete member was seated on shims on the column below so as to leave a 20-30mm gap. This gap was to be grouted in the same operation as the grouting of the vertical column bars in the corrugated ducting in the precast member. Then, the perimeter of the joint between the precast concrete beam and the concrete column below was sealed, leaving 16mm diameter outlets fitted to plastic hose at each corner of the joint.

3.4.5 Concreting of the Cast in Place Connections

In Units 1 to 4, the concrete in the connection at midspan was vertically placed. Unfortunately the compressive strength of the concrete in the midspan connection between the precast concrete elements of Unit 1 was below acceptable strength and had to be removed and recast. For all other units the cast in place concrete was of acceptable strength.
In Unit 5 the concrete had a high slump and the texture of the surface of the top horizontal construction joint indicated that bleeding of the fresh concrete had occurred. Cracks in the concrete along the protruding stirrups of the precast beams were also observed. The horizontal construction joint at the column was wired brushed and water blasted to obtain a rather rough appearance. Then the top column was cast from the top from a hole left in the top steel plate.

The joint region in Unit 6 was saturated with water for at least 6 hours prior grouting. An attempt was made to pump a none-grown described in Section 3.5.2.2 grout through the bottom of the joint but it segregated and packed inside the pump and the hose and it was impossible to carry on with the operation. Instead, the grout was fed by gravity using a tremie hose located in a corner duct. Fig. 3.22 illustrates this operation. The grout was seen to ascend through each duct and through the four corner hoses at the interface between the precast beam and the column below. Topping was required after the operation because of losses on the hydraulic head. It was felt that most of the coarser material remained in the bottom and did not ascend through the corrugated ducts and therefore the topping added would introduce a more homogeneous material. Inspection of the grout in the plastic hoses showed that some channelling had occurred because of the migration of the free water to the top of the hose. This observation was also made for the dummy ducting cast at the same time as the connection. The first 50mm of the grout in the ducts was removed because of the excessive bleeding observed. Then the top column was cast following the same procedure as for Unit 5.

3.5 MATERIALS

3.5.1 Concrete

The concrete was provided by a commercial ready-mix supplier with a specified target 28 day compressive strength of 30MPa for all units except Unit 4 where 35MPa concrete was specified. In general the maximum specified aggregate size was 20mm but in some cases 13mm aggregate was used because of the congestion of the reinforcement cage. Table 3.2 summarizes the main concrete properties. 100mm diameter by 200mm high cylinders were used to determine the mechanical properties of the concrete. The samples were cast over a vibrating table, cured in a fog room at 20°C constant temperature and 100% relative humidity and left to dry for at least four hours before testing. The compression strengths in Table 3.2 are the average of three tests while the tensile strengths are the average of two splitting tests. All tests were quasi-statically conducted in an 2500kN Avery Universal Testing Machine.

3.5.2 Grout

3.5.2.1 General

A study of different properties of fluid high strength grouts needed for joining the precast concrete elements in Unit 6 was undertaken in this project.
Three different cement-based grouts were investigated; one of them was designed in the concrete laboratory at the University of Canterbury while the other two were commercially available preparatory grouts.

The main parameters to be looked at were fluidness, segregation and bleeding of the fresh grout. In addition, the compressive and splitting strength, the elastic modulus, the size effect of the test cylinders and the enhancement of the compressive strength due to the metallic ducting were investigated.

### 3.5.2.2 Self-Mixed Grout

Several trials were prepared in the concrete laboratory to obtain an ideal grout with a compressive strength at 28 days, measured on 50mm diameter by 100mm high cylinders, of at least 10MPa higher than the nominal compressive strength of 30MPa of the precast concrete unit to be grouted. The fluidness of the grout was measured according to the flow cone method [ASTM Standard C939-87 (1988)]. It was found that the ideal flow should be kept around 40-50 seconds to be able to spread freely through the construction joint of the precast unit while avoiding excessive segregation.

Table 3.3 contains the final proportions per cubic metre of the mix as poured into Unit 6. Kaiapoi fine sand with a fineness modulus of 2.15, a maximum aggregate size of 5mm and a specific gravity of 2.59 was used as aggregate in the grout. Onoda, an expansive agent, was added to compensate for shrinkage caused by the high content of water and cement in the mix. In addition, Daracem 100 was incorporated to reduce as much as possible the water in the mix. This product also had a retardant effect.

### 3.5.2.3 Preparatory Grout

The preparatory grouts selected for study were SIKA 212 and Monogrupt of Grace. The grouts were mixed with the maximum amount of water recommended by the manufacturer of 4.5 and 4.6 lts per 25 kg bag for SIKA 212 and Monogrupt respectively.

Two series of tests were conducted for each grout. SIKA 212 showed a low fluidness with a time in the cone test of 62 and 73 seconds. The consistency of the mix was acceptable although some segregation was observed. Monogrupt showed a low viscosity. The time in the cone test was 17 and 33 seconds. Segregation, large plastic settlement and bleeding were observed in this grout.

### 3.5.2.4 Test Results

Table 3.4 shows the grout compressive strength of the slices cut from a 66mm diameter by 700mm high dummy duct that was vertically cast following actual construction practice. The aspect ratio of the slices was kept as close as possible to 2:1.
<table>
<thead>
<tr>
<th>Pour No.</th>
<th>Location</th>
<th>Maximum Aggregate Size (mm)</th>
<th>Slump (mm)</th>
<th>Age at Test (Days)</th>
<th>Compressive Strength $f'_c$ (MPa)</th>
<th>Split Cylinder Tensile Strength $f'_t$ (MPa)</th>
<th>$f'_t/f'_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>At 28 Days</td>
<td>At Test</td>
<td>At 28 Days</td>
</tr>
<tr>
<td>1</td>
<td>Unit 1, precast concrete member</td>
<td>20</td>
<td>85</td>
<td>116</td>
<td>25</td>
<td>29</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>Unit 1, connection</td>
<td>13</td>
<td>100</td>
<td>64</td>
<td>32</td>
<td>41</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>Unit 2, precast concrete member</td>
<td>20</td>
<td>90</td>
<td>168</td>
<td>33</td>
<td>33</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>Unit 2, connection</td>
<td>20</td>
<td>50</td>
<td>48</td>
<td>31</td>
<td>32</td>
<td>2.6</td>
</tr>
<tr>
<td></td>
<td>Unit 3, precast concrete member</td>
<td>20</td>
<td>180</td>
<td>21</td>
<td>28</td>
<td>29</td>
<td>3.0</td>
</tr>
<tr>
<td>5</td>
<td>Unit 3, connection</td>
<td>20</td>
<td>80</td>
<td>100</td>
<td>31</td>
<td>35</td>
<td>3.5</td>
</tr>
<tr>
<td>6</td>
<td>Unit 4, precast concrete member</td>
<td>13</td>
<td>110</td>
<td>21</td>
<td>36</td>
<td>36</td>
<td>3.5</td>
</tr>
<tr>
<td></td>
<td>Unit 5, precast beams, bottom column</td>
<td>20</td>
<td>207</td>
<td>207</td>
<td>36</td>
<td>41</td>
<td>3.5</td>
</tr>
<tr>
<td>7</td>
<td>Unit 4, repair region</td>
<td>13</td>
<td>75</td>
<td>32</td>
<td>62</td>
<td>62</td>
<td>4.5</td>
</tr>
<tr>
<td>8</td>
<td>Unit 6, precast beam, bottom column</td>
<td>20</td>
<td>100</td>
<td>183</td>
<td>36</td>
<td>44</td>
<td>3.2</td>
</tr>
<tr>
<td>9</td>
<td>Unit 5, beam-column joint, beam top</td>
<td>20</td>
<td>200</td>
<td>62</td>
<td>20</td>
<td>27</td>
<td>2.3</td>
</tr>
<tr>
<td>10</td>
<td>Unit 5, top column</td>
<td>20</td>
<td>90</td>
<td>61</td>
<td>32</td>
<td>43</td>
<td>3.2</td>
</tr>
<tr>
<td>11</td>
<td>Unit 6, top column</td>
<td>20</td>
<td>150</td>
<td>20</td>
<td>35</td>
<td>35</td>
<td>2.8</td>
</tr>
</tbody>
</table>

Table 3.2
Concrete Properties
### Table 3.3
Mix Proportions of Self-Mixed Grout

<table>
<thead>
<tr>
<th>COMPONENT</th>
<th>AMOUNT per m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water</td>
<td>290 kgf</td>
</tr>
<tr>
<td>Ordinary Portland Cement</td>
<td>547 kgf</td>
</tr>
<tr>
<td>Onoda (Expanding Agent)</td>
<td>68 kgf</td>
</tr>
<tr>
<td>Sand</td>
<td>1333 kgf</td>
</tr>
<tr>
<td>Daracem-100 (Superplastizicer)</td>
<td>4377 ml</td>
</tr>
</tbody>
</table>

### Table 3.4
Compressive Strength $f'_c$ (MPa), at 28 Days of Grouted Cylinder Cut from Dummy Ducts

<table>
<thead>
<tr>
<th>Position</th>
<th>Self-Mixed</th>
<th>SIKA 212 (a)</th>
<th>SIKA 212 (b)</th>
<th>Monogrouting (a)</th>
<th>Monogrouting (b)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top</td>
<td>36.0</td>
<td>45.3</td>
<td>56.6</td>
<td>66.1</td>
<td>40.3</td>
</tr>
<tr>
<td></td>
<td>36.6</td>
<td>59.8</td>
<td>67.3</td>
<td>59.8</td>
<td>40.5</td>
</tr>
<tr>
<td></td>
<td>44.8</td>
<td>55.9</td>
<td>65.7</td>
<td>66.1</td>
<td>46.6</td>
</tr>
<tr>
<td></td>
<td>40.3</td>
<td>67.3</td>
<td>55.9</td>
<td>70.7</td>
<td>49.2</td>
</tr>
<tr>
<td></td>
<td>48.0</td>
<td>65.2</td>
<td>63.8</td>
<td>59.3</td>
<td>52.3</td>
</tr>
<tr>
<td>Bottom</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>41.1</td>
<td>58.7</td>
<td>61.9</td>
<td>64.4</td>
<td>45.8</td>
</tr>
</tbody>
</table>

### Table 3.5
Mean Compressive Strength of Grout, $f'_c$ (MPa)

<table>
<thead>
<tr>
<th>Nominal Size of Cylinder (mm)</th>
<th>Self-Mixed</th>
<th>SIKA 212 (a)</th>
<th>SIKA 212 (b)</th>
<th>Monogrouting (a)</th>
<th>Monogrouting (b)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50x100</td>
<td>63.3</td>
<td>62.5</td>
<td>68.3</td>
<td>65.0</td>
<td>60.6</td>
</tr>
<tr>
<td>66x132</td>
<td>-</td>
<td>49.9</td>
<td>63.0</td>
<td>61.6</td>
<td>-</td>
</tr>
<tr>
<td>100x200</td>
<td>-</td>
<td>51.2</td>
<td>63.4</td>
<td>61.0</td>
<td>55.0</td>
</tr>
</tbody>
</table>
Table 3.6
Splitting Strength of Grout

<table>
<thead>
<tr>
<th>Nominal Size of Cylinder (mm)</th>
<th>SIKA 212 (a)</th>
<th>SIKA 212 (b)</th>
<th>Monogroup (a)</th>
<th>Monogroup (b)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$f'_g$ (MPa)</td>
<td>$f'_g$ (MPa)</td>
<td>$f'_g$ (MPa)</td>
<td>$f'_g$ (MPa)</td>
</tr>
<tr>
<td>50x100</td>
<td>3.8</td>
<td>4.3</td>
<td>4.9</td>
<td>4.1</td>
</tr>
<tr>
<td></td>
<td>0.46</td>
<td>0.52</td>
<td>0.60</td>
<td>0.53</td>
</tr>
<tr>
<td>100x200</td>
<td>3.3</td>
<td>3.3</td>
<td>3.2</td>
<td>4.5</td>
</tr>
<tr>
<td></td>
<td>0.47</td>
<td>0.41</td>
<td>0.40</td>
<td>0.60</td>
</tr>
</tbody>
</table>

Table 3.7
Initial Elasticity Modulus of Grout

<table>
<thead>
<tr>
<th>Grout</th>
<th>$E_g$ (GPa)</th>
<th>$E_g/f'_g$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Self-Mixed</td>
<td>26.2</td>
<td>3293</td>
</tr>
<tr>
<td>SIKA (b)</td>
<td>29.7</td>
<td>3588</td>
</tr>
<tr>
<td>Monogroup (a)</td>
<td>22.5</td>
<td>2790</td>
</tr>
</tbody>
</table>

It can be noted that in all cases in Table 3.4 the compressive strengths trend to increase with the depth of the cylinder except in the series (a) of Monogroup. One reason for this behaviour is believed to be caused by free water ascending through the duct and causing a high water cement ratio in the top segments. Another factor could be the effect of cutting on the edges of the cylinders. Results obtained in this study do not permit to withdraw conclusions regarding the variation of the grout compressive strength throughout the height of the duct.

The compressive strength of the grout was slightly affected by the size of the cylinder test as shown in Table 3.5. Test results, presented are the mean of 5 cylinder tests for 50 and 100mm diameter cylinders and 2 tests for the 66mm diameter probes. It was observed that the larger the cylinder test the smaller its mean compressive strength between 50mm diameter cylinders and 68mm or 100mm diameter test cylinders. No differences were observed between the 68mm and 100mm diameter cylinders. From the results shown in Table 3.5 it can be concluded that size effects can be
ignored for the ratio between the diameter of the ducts normally used in precast concrete construction typical of System 2 and the diameter of the test cylinders used for determining the compressive strength of the grout.

The splitting strength of the grout $f_{sp}$, taken as the mean of three tests is shown in Table 3.6. The ratio between the splitting strength and its compressive strength varies similarly to concrete.

Table 3.7 shows the measured modulus of elasticity of different grouts estimated from measurements diametrically taken over a gauge length of 50.8mm on 50mm diameter by 100mm high cylinders. The values presented are the mean of three tests in which the initial modulus was measured as the slope of the secant line passing through the origin and the strain at a stress of 0.5 $f'_{c}$. It is evident that the initial modulus is significantly lower than the modulus of $4,700\sqrt{f'_{c}}$ recommended by the Concrete Code [NZS 3101 (1982)] for normal weight concrete.

The final variable to be studied was the enhancement of the compressive strength of the grout caused by the confinement provided by the 0.4mm thick metallic ducting to be used in Unit 6. Fig. 3.23 plots the mean unconfined versus confined compressive strength of different grouts as measured on two 66mm diameter by 132mm high cylinder tests. Also shown in Fig. 3.23 is the predicted enhancement according to the formula postulated for concrete by Richart et al [Park and Paulay (1975)].

\[
f'_{cc} = f'_{c} + 4.1 f_{t} \quad (3.1)
\]

where $f'_{cc}$ is the confined compressive strength of concrete and $f_{t}$ is the lateral confining pressure. It can be seen that Eq. 3.1 fits reasonably well the data obtained in this study and therefore appears to be also applicable for determining the confined strength of grout.

![Fig. 3.23 - Confined versus Unconfined Compressive Strength of Grout.](image)
In conformity with the New Zealand practice, the longitudinal reinforcement was deformed steel bar with a lower characteristic yield strength of 300MPa for the beams and 430MPa for the columns. Plain round steel bar with a lower characteristic yield strength of 300MPa was used for transverse reinforcement.

Table 3.8 presents the mechanical tensile properties of the steel used in the test units. Test results are the average of two tests monotonically conducted at a quasi-static rate in an 1000kN Avery Universal Testing Machine. Strains up to approximately 4% were measured with a Batty Gauge Extensometer with a gauge length of 50.8mm. The resolution of this device was 25με. The ultimate strain, ε_u, could not be directly measured since the Batty Gauge had been previously removed from the test specimen. Instead the apparent ultimate strain, ε_u,a, was obtained after unloading the test sample upon maximum loading as illustrated in Fig. 3.24. A Mitutoyo Digital Vernier with a resolution of 0.01mm was used to measure the extension between two initial gauge lengths of 100mm in the original specimen. However, it is possible to obtain a reasonable approximation of the ultimate strain by accounting for the unloading slope. All ultimate strains in Table 3.8 have been calculated as ε_u = ε_u,a + f_u / 0.8E_s. The coordinates ε_{sh,1}, f_{sh,1} are also presented to describe the strain hardening region according to the model discussed in Chapter 2.

All steels showed a well defined yield plateau region, however the yield strength of the D28 bars used in Units 1, 2 and 5, which were supposed to come from the same batch, varied and tended to two different values. Results from eight tests indicate that it is likely that steel from two different heats was delivered to the project.

Fig. 3.24 - Typical Stress-strain Diagram for Reinforcing Steel.
3.6 **TEST PROCEDURE**

The activities before commencing testing included a coat of water based white paint or a combination of white and light blue paint applied to the units to permit better crack observation. Besides, all strain gauges were checked for continuity and resistance to earth; those with a resistance of less than $2\,\Omega$ were rejected. All linear potentiometers were calibrated in position in the test unit using steel spacers. Clip gauges that were fabricated from a prototype supplied by the Department of Civil Engineering at the University of Auckland [Fenwick and Thom (1982)] were calibrated with a metric calibrator and then fitted in position in the test specimen. All calibration data was checked with a linear regression program and only data with a coefficient of correlation larger than 0.99998 was accepted. The principle of these gauges is the same as for the clip gauges used for testing the reinforcing steel specimens discussed in Chapter 2. At the fixing points at the bases of the columns of the clip gauges, miniature bearings were inserted to ensure a roller type of boundary condition. At this point 4mm diameter steel rods held the clip gauges.

The test was quasi-statically conducted with load applied in small increments. At each increment a selected set readings was taken with the data logger unit and at the peak of each run a complete set of readings, including manual readings and crack widths, was recorded. At this point photographs of the test specimen were also taken. Prior to unloading, another complete set of readings was taken, to pick up any error in the manual readings and any creep of the strain gauges.

3.7 **TEST SEQUENCE**

As is common with many other quasi-static seismic tests, the test regime of displacement history imposed on the test units did not attempt to follow the displacement sequence from the response of a structure to a specific earthquake record. Instead, a simple test sequence with increasing symmetrical cyclic displacements was adopted, as is common in tests conducted to evaluate the probable seismic resistance of a sub-component. The rationale behind the simple test sequence is that a specimen that behaves satisfactorily when subjected to appropriately severe symmetrical cyclic displacements is likely to behave satisfactorily during a real seismic event [Park (1989)].

The quasi-static test sequence imposed on the test units is shown in Fig. 3.25. The two first cycles, runs 1 to 4, were load controlled; the rest of the test was displacement controlled.

The load controlled cycles were used to determine the initial yield displacement as well as the initial "elastic" stiffness of the units. The specimens were loaded to $3/4$ of the theoretical lateral load capacity, $H_a$, which was calculated using the measured properties of the materials and assuming a perfect elasto-plastic stress-strain relation for the reinforcing steel and an equivalent rectangular block compressive stress for concrete. The corresponding interstorey horizontal displacements (measured at column tops of units) so measured during these runs were averaged to obtain the displacement $\Delta_{\text{st}}$ associated with the application of load $0.75H_a$. The first yield displacement, $\Delta_y$, was linearly extrapolated as:
Table 3.8
Mechanical Properties of Reinforcing Steel of Test Units

<table>
<thead>
<tr>
<th>Description</th>
<th>Location</th>
<th>$f_y$ (MPa)</th>
<th>$\varepsilon_{sh}$ (%)</th>
<th>$E_s$ (GPa)</th>
<th>$\varepsilon_{sh,1}$ (%)</th>
<th>$f_{sh,1}$ (MPa)</th>
<th>$\varepsilon_{se}$ (%)</th>
<th>$f_{se}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R6</td>
<td>Unit 4</td>
<td>364</td>
<td>1.2</td>
<td>179</td>
<td>3.0</td>
<td>402</td>
<td>5.7</td>
<td>428</td>
</tr>
<tr>
<td>R10</td>
<td>All units</td>
<td>356</td>
<td>1.6</td>
<td>202</td>
<td>3.0</td>
<td>394</td>
<td>17.4</td>
<td>469</td>
</tr>
<tr>
<td>R12</td>
<td>Unit 3</td>
<td>324</td>
<td>2.2</td>
<td>201</td>
<td>4.0</td>
<td>382</td>
<td>15.3</td>
<td>449</td>
</tr>
<tr>
<td>R12</td>
<td>Unit 4</td>
<td>317</td>
<td>2.0</td>
<td>200</td>
<td>3.0</td>
<td>355</td>
<td>15.3</td>
<td>436</td>
</tr>
<tr>
<td>R16</td>
<td>Units 1, 4, 5 and 6</td>
<td>298</td>
<td>2.5</td>
<td>206</td>
<td>4.0</td>
<td>349</td>
<td>24.7</td>
<td>444</td>
</tr>
<tr>
<td>D20</td>
<td>Unit 3</td>
<td>307</td>
<td>2.3</td>
<td>196</td>
<td>3.8</td>
<td>352</td>
<td>17.5</td>
<td>447</td>
</tr>
<tr>
<td>D24</td>
<td>Unit 4</td>
<td>320</td>
<td>2.1</td>
<td>178</td>
<td>4.0</td>
<td>381</td>
<td>23.2</td>
<td>477</td>
</tr>
<tr>
<td>D24</td>
<td>Units 5 and 6</td>
<td>285</td>
<td>1.9</td>
<td>180</td>
<td>4.0</td>
<td>352</td>
<td>23.7</td>
<td>444</td>
</tr>
<tr>
<td>D28</td>
<td>Units 1, 2 and 4</td>
<td>313</td>
<td>2.1</td>
<td>202</td>
<td>4.1</td>
<td>373</td>
<td>21.1</td>
<td>477</td>
</tr>
<tr>
<td>D28</td>
<td>Unit 5</td>
<td>321</td>
<td>2.1</td>
<td>186</td>
<td>3.8</td>
<td>380</td>
<td>21.5</td>
<td>481</td>
</tr>
<tr>
<td>HD20</td>
<td>Units 1, 2, 3 and 5</td>
<td>456</td>
<td>1.4</td>
<td>193</td>
<td>3.9</td>
<td>542</td>
<td>15.3</td>
<td>617</td>
</tr>
<tr>
<td>HD24</td>
<td>Units 5 and 6</td>
<td>486</td>
<td>1.8</td>
<td>195</td>
<td>4.0</td>
<td>570</td>
<td>14.6</td>
<td>637</td>
</tr>
<tr>
<td>HD28</td>
<td>Unit 4</td>
<td>440</td>
<td>1.6</td>
<td>193</td>
<td>4.0</td>
<td>527</td>
<td>18.8</td>
<td>603</td>
</tr>
<tr>
<td>30x10 mm straps</td>
<td>Unit 4</td>
<td>315</td>
<td>1.9</td>
<td>201</td>
<td>3.0</td>
<td>358</td>
<td>20.0</td>
<td>479</td>
</tr>
<tr>
<td>36x0.4 mm metal</td>
<td>Unit 6</td>
<td>290</td>
<td>2.7</td>
<td>190</td>
<td>-</td>
<td>-</td>
<td>20.0</td>
<td>361</td>
</tr>
</tbody>
</table>
and the "elastic" stiffness, $K_e$:

$$K_e = \frac{3}{4} \frac{h}{\Delta_{y_5}}$$  \hspace{1cm} \text{(3.3)}

The imposed displacement cycles were applied following runs 1 to 4 to the levels of nominal displacement ductility factors, $\mu_\Delta$, shown in Fig. 3.25, where

$$\mu_\Delta = \frac{\Delta}{\Delta_y}$$ \hspace{1cm} \text{(3.4)}

where $\Delta$ is the applied interstorey horizontal displacement. A similar procedure was also used in the data reduction to define the curvature and rotational ductility factors.

---

Fig. 3.25 - Test Sequence of Cyclic Load and Displacement Used for Tests.
The smaller ideal lateral capacity of Unit 1 was mistakenly taken instead of the "actual" theoretical lateral capacity and caused a 6.7% error in definition of the first yield displacement and in the displacement ductility factor. However, it is believed that this error does not compromise the results obtained from this test. In order to compare the behaviour Unit 6 was subjected to the same displacement history as Unit 5.

The end of the test was reached when the lateral load in the run dropped to less than 80% of the maximum applied load recorded in the initial runs. The available displacement ductility factor was taken equal to be \( \mu_\alpha = \Sigma \mu_\alpha / 8 \) [Park (1989)] but no larger than 8, where \( \Sigma \mu_\alpha \) is the cumulative displacement ductility factor attained in the runs prior to failure.

3.8 INSTRUMENTATION

3.8.1 Measurement of Loads

The loads measured were the forces applied with all the hydraulic actuators to all units, and the reactive forces at the beam ends of Units 5 and 6. Load cells coupled with the hydraulic actuators consisted on a hollow cylinders machined from high strength steel and containing strain gauges. Two independent full bridge (Poisson) circuits were set up for each load cell. Each arm of the circuit had 2-5mm strain gauges of 350Ω of resistance. One circuit was directed to a strain indicator and the other to a data logger unit. The load cells had been calibrated in compression in an Avery Universal Testing Machine. It was assumed that the tensile characteristics of the load cells were equal to those measured in compression.

The RHS sections at the end of the beams of Units 5 and 6 (see loading frame LF2 in Fig. 3.10 (a)) were instrumented with 120Ω 5mm strain gauges forming a full bridge (Poisson) and calibrated in the same Universal Testing Machine as used for the load cells.

3.8.2 Measurement of Displacements and Deformations

3.8.2.1 Measurement of Displacements

The displacements measured during the test programme were the gross horizontal displacement, the rigid body horizontal displacement and the lateral displacement due to rigid body rotation of the test specimen. These sources of displacements were measured with Sakae linear potentiometers of 10kΩ of resistance. Figs. 3.26 and 3.27 show the position of these devices on the test specimen. The two linear potentiometers, with 300mm of travel, marked A, measured the gross horizontal displacement of the column top and its twist at the pin height. A steel wire, a pulley and a weight were used to connect the devices from the specimen to a reference point located outside the strong floor area. Linear potentiometers, with 15mm travel, marked B, monitored the horizontal rigid body displacement of the units at the pin in the bottom column due to clearances between the sleeve and the pin. The linear potentiometers marked C and C', with 15mm of travel, enabled the calculation of
Fig. 3.26 - Lay out of Linear Potentiometers Measuring Displacements of Units 1 to 4.

Fig. 3.27 - Lay out of Linear Potentiometers Measuring Displacements of Units 5 and 6.
the rigid body rotation. In the test of Unit 1 linear potentiometers marked B, C and C' were mounted on steel frames attached to the test floor. Measurements taken with a theodolite indicated that displacement readings gathered were affected by deformation within the testing slab floor caused by the axial forces transmitted by the loading system. Hence, in the remainder of the test programme, these linear potentiometers were attached to steel frames placed away from the test floor. Because of space constraints, the linear potentiometers marked C and C' were not in line with the pins at the ends of the beams of Units 5 and 6.

The net interstorey displacement was determined using the information gathered from these transducers as follows:

\[ \delta_{H} = \delta_{A} - \delta_{B} - (\delta_{C} - \delta_{C'}) \frac{\ell}{\ell_{h}} \]  \hspace{5cm} (3.5)

where \( \delta_{H} \) is the net horizontal column or interstorey displacement; \( \delta_{A}, \delta_{B}, \delta_{C} \) and \( \delta_{C'} \) are the displacements measured by the transducers marked A, B, C and C' respectively; \( \ell_{c} \) is the vertical distance between the column pin and \( \ell_{h} \) is the horizontal distance between the linear potentiometers marked C and C'. The 50mm travel linear potentiometers marked D and D' in Fig. 3.26 were used to maintain the same horizontal displacement (interstorey drift) between both columns.

3.8.2.2 Measurement of Internal Deformations

Several devices were utilized in the test programme to monitor the internal deformations in the test specimens. Figs. 3.28 and 3.29 show the location and type. The deformations of the chords of the beams was measured using 30 or 50mm travel Sakae linear potentiometers of 10kΩ of resistance. They were mounted on steel brackets screwed into the 10mm steel rods embedded in the concrete. The readings from the top and bottom linear potentiometers were used to determine the fixed-end rotation in the beams of all units except Unit 4. The remaining transducers were used to determine the rotation of the chord and the average strains. This procedure will be discussed later in this chapter. The accuracy of measurements using this method was satisfactory. However it was always observed that near the end of the tests the steel rods kinked in the diagonal cracks and tilted, inducing false readings on the devices. At this stage it was decided to remove them from the test specimen.

The mid-depth elongation of the beams was directly monitored by 200mm travel linear potentiometers attached to the ends of the test specimens by a steel wire and a pulley. In the beams of Units 5 and 6 it was not possible to attach the steel wire at the mid-depth of the beam. Instead both attachments were located near the bottom chord. In addition, the linear potentiometers located in the chords of the beams were used as an indirect way of determining this elongation.

A partial measure of the rotation of the columns of Unit 5 and 6 was estimated from manual readings using DEMEC (demountable mechanical) gauges. Any displacement was read between drilled steel targets that had been waxed to the surface of the concrete. Similarly, DEMEC gauges were used
Fig. 3.28  Lay out of Linear Potentiometers Measuring Internal Deformations of H-shaped Units.
Fig. 3.29 - Lay out of Linear Potentiometers Measuring Internal Deformations of Unit 5 and 6.

to monitor any relative sliding shear between the horizontal construction joints in the columns of these two units. The position of DEMEC gauges is shown in Fig. 3.29.

Diagonal deformations of the beam span and in the beam-column joint region were made with the intention of monitoring the average shear distortions. An extended DEMEC gauge was initially used for this measurement in Units 1 to 3. A further development was the use of clip gauges and linear potentiometers mounted on stiff steel brackets to replace most diagonal readings manually taken. At large shear distortions most of the devices used for reading the diagonal deformations ran out of travel and instead manual measurements were taken using a 1m steel rule, which permitted readings within 0.5mm of accuracy.

3.8.3 Reinforcement Strains

3.8.3.1 Local Strains

Local strains in the reinforcing bars were measured using 120Ω 5mm foil strain gauges type Showa N11-FA-5-120-11 with a nominal gauge factor of 2.1. Details of the position of the strain gauges on the reinforcing bars will be given with the test results.

The surface preparation of the bars before attaching the strain gauges had the following steps:
a) The deformation of the bar where the gauge was to be attached was removed using either a file or a pneumatic belt sander. Care was taken to avoid an excessive reduction of the bar section in this region.

b) The prepared surface of the bar was then abraded by cross hatching at 45° using a 180 grit wet and dry paper.

c) The bar surface was then decontaminated using Methyl-Ethyl-Ketone, (MEK) applied with cotton-tipped applicators. The surface was scrubbed into the surface until the applicator appeared to have removed all contaminants. Caution was taken to avoid any material, such as grease, to recontaminate the surface.

Each strain gauge was stuck to a piece of sellotape with the backing surface exposed. Then, Loctite 401, an ethyl cyanoacrylate adhesive, was spread in a thin layer to cover all the backing surface of the strain gauge and then the gauge was immediately placed on the reinforcing bar. The strain gauge was held with pressure for space of at least one minute and the sellotape was removed several minutes later. Self-adhesive terminals were placed near the strain gauge to complete the circuit. A layer of Shinkoh SN/4, a waterproofing cement, was placed within 24 hours over the strain gauges and terminals. At least two additional layers of the waterproofing compound were added. Finally, 3M VM vynil mastic tape was used to cover the instrumented region and give further insulation and protection against physical damage.

The wires from the terminals were fed through a 4mm diameter PVC flexible sleeve to insulate them from moisture and give mechanical strength in the region near the terminal where the wires had to be tied to the reinforcing bar. They were bundled in groups containing not more than 8 sleeves and directed to a region in the test specimen that would not interfere with the instrumentation.

The reliability of the strain gauge readings was assessed in three cyclic load tests on machined reinforcing steel specimens similar to those used in the test program discussed in Chapter 2. Four strain gauges were arranged longitudinally at the centre of the specimens. The surface preparation followed the steps described above except for one strain gauge in each test where the surface was not abraded. The cyclic load tests stopped for five minutes at each load reversal to simulate the conditions at the peak of each run in tests on precast concrete assemblages in this programme.

Fig. 3.30 shows the test results. The main differences in the strain readings were due to the Lüders bands at the initiation of yielding which do not spread uniformly across the section of the specimen. Creep was always observed before the failure of the strain gauges. This creep occurred usually during the lapse before reversal of load direction. The strain gauges glued on to a smooth unabraded surface performed surprisingly poorly. Although the other strain gauges showed a longer life, it can be said that reliable strain readings beyond 1.2% to 1.8% strain cannot be obtained. Microscopic observations showed that the failure at large strains was due to debonding of the strain gauges at the surface of the specimen. The strain gauges themselves remained in working condition. It appears that creep during the lapse taken at the peak of each loading run for crack observation,
Fig. 3.30 - Results of Cyclic Load Tests to Assess the Reliability of Strain Gauges Used.
manual readings, etc. may show a critical condition for the strain gauge, especially if they are located in regions of moderate to high plasticity. It also appears that after the strain gauge debonds, the gauge is still capable of some cyclic response, because of the waterproofing cement which is still firmly adhered to the specimen and the strain gauge. The author has observed that because of this phenomena some researchers in the past have not realized that some strain gauges have failed and have used these false readings as "measured" data. It appears that the best way to identify whether a strain gauge has failed is to compare its reading at the peak of each loading run with that obtained from a scan prior unloading. Also, local bar strains measured by strain gauges can be affected by factors such as tension stiffening and local kinking of the bars near a crack. Hence, some judgement is required when analyzing strain readings obtained from electrical resistance strain gauges.

An additional problem faced in this project was the measurement of strains in the beam bars passing through the beam-column joint region in Units 5 and 6. The wires attached to the strain gauges can be subjected to large strains, which eventually may cause the loss of the circuit, because of the movement of the instrumented bars relative to the surrounding concrete. Hence, the region where the strain gauges were attached to the beam bar was covered with a soft material to give some flexibility to the wire near the terminal. Clip gauges were attached to these bars at the face of the columns since it was expected that the life of the strain gauges would not last beyond the initial stages of the test. The author believes that some other techniques could be successfully implemented to give more reliability to strain measurements in this important region. For instance, the use of spot welded strain gauges could be studied.

3.8.3.2 Average Strains

An alternative to measuring local strains using electrical resistance strain gauges is to measure average reinforcement strains using a mechanical strain gauge. In this study average reinforcement strains were initially measured using DEMEC strain gauges with a resolution of 0.002mm/division. That is, 2 and 1 με of resolution for gauge lengths of 102 and 204mm, respectively. Drilled steel targets were waxed onto the heads of screws that were locked to the steel studs that had been welded to the reinforcement. DEMEC gauges were later replaced by clip gauges that could be automatically scanned by the data logger unit. Clip gauges hanging from 4mm diameter rods were fixed to the welded studs using T-shaped fasteners.

Average strains at the level of the longitudinal reinforcing steel could also be estimated using the readings obtained from the linear potentiometers placed along the chords of the beams. If a diagonal crack is idealized at the centre of each segment gauged by a linear potentiometer as shown in Fig. 3.31, then the average strain average strain, $\varepsilon_{avg}$, at the level of the longitudinal reinforcing steel is given by

$$\varepsilon_{avg} = \frac{\delta}{L}$$

where
Fig. 3.31 - Average Strain at the Level of the Longitudinal Reinforcement Obtained from Linear Potentiometer Readings.

Fig. 3.32 - Set-up of Displacement Transducers to Monitor the Bar Slippage in the Beam-Column Joint Region of Units 5 and 6.
\[
\delta_r = (d - d') \frac{\delta_s}{d_p}
\]

and \(\ell_g\) is the initial gauge length, \(d - d'\) is the distance between top and bottom beam reinforcement, \(\delta_r\) is extension measured with the linear potentiometer over gauge length \(\delta_s\), \(d_p\) is the distance between the linear potentiometer and the centroid of the steel in compression.

Note that a small error in the idealization of the cracks is introduced when it is assumed that diagonal cracks will radiate into the beam from the column face up to a distance larger than the critical distance \(x_{cr}\) found using Eq. 3.12.

The average strains so found will not, of course, be equal to those in the reinforcing steel itself. However, they will follow the same trends and the error when the reinforcing steel is in tension is not expected to be large. Moreover, average strains at large displacement ductility cycles found using Eq. 3.6 were considered to give more reliable information than those obtained directly from strain gauge measurements. This is of special interest in Units 3, 4, 5 and 6 where average reinforcement strains were not directly measured along the beam span using either DEMEC gauges nor clip gauges.

### 3.8.4 Bar Slippage

The slippage of the longitudinal bars passing through the beam-column joint region of Units 5 and 6 was monitored as shown in Fig. 3.32. A 30mm travel linear potentiometer was used to measure the relative displacement at the column centreline between the concrete surface and a target stud welded to the D24 reinforcing bars. Clip gauges were used to measure the elongation of the bars between this target and targets welded on the same bars at 10mm away from the column faces. The so called local bar slip at the centreline of the column is based on the assumption that the concrete in this region forms an infinite rigid matrix around the bar. By adding the local bar slip at the column centre line to the elongation of the bars at 310mm from this point it is then possible to determine the movement of the bars relative to the column centre line. For obvious reasons this movement is not a local bar slip, because the concrete in the beam can not be considered as forming an infinity rigid matrix. However, the bar slip at the level of the outer column bars can be approximated by subtracting the elongation of the bar due to the strain to the measured bar movement. For that, it is necessary to assume that the beam bar strain between the outer column bars and the target at 10mm from the column face is constant and is equal to the one measured with the clip gauges over a gauge length of 145mm (see Fig. 3.32). This assumption is reasonably accurate since the bond conditions in this region deteriorate as the test progresses.

### 3.8.5 Data Acquisition System

All circuits from load cells, linear potentiometers, clip gauges and strain gauges were connected to data logger units with analogue-to-digital converter cards that converted voltage changes into divisions ranging between ±2048. The output voltage of each transducer was amplified several times to obtain a predetermined sensitivity. Table 3.9 summarizes input voltages, gain factors and
resolution of the transducers used. Two data logger units were used in the test programme. Unit 1 was tested using a Cedacs I unit with a characteristic voltage of 20000mv/4096 divisions. This data logger unit presented problems with electrical noise as well as with wild points. The software to run this data logger was basic and during the test, displacements and load had to be manually reduced. A Burr Brown data logger unit replaced the Cedacs I unit in the remainder tests. Its characteristic voltage was equal to 10000mv/4096 divisions. The tailored software was especially written to give displacements and forces in real time.

The second independent circuit of the load cells was read by a strain indicator. For Unit 1 an analog Showa SI-10 strain indicator connected to a switch box was used. For the other tests Measurements Group P-3500 digital strain indicators were selected.

The voltage change of the linear potentiometers marked A and the differential output of linear potentiometers D and D' in Figs. 3.26 and 3.27 were monitored using a Hewlett-Packard Digital Voltmeter model 3440A.

<table>
<thead>
<tr>
<th>Transducer</th>
<th>Input Voltage (v)</th>
<th>Bridge Configuration</th>
<th>Gain Factors</th>
<th>Nominal Resolution</th>
<th>Accuracy</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Cedacs I</td>
<td>Burr Brown</td>
<td></td>
</tr>
<tr>
<td>Load Cells</td>
<td>4</td>
<td>full</td>
<td>250</td>
<td>500</td>
<td>1.05 dlu/kN (1)</td>
</tr>
<tr>
<td>RHS Sections</td>
<td>4</td>
<td>full</td>
<td>-</td>
<td>200</td>
<td>1.35 dlu/kN (2)</td>
</tr>
<tr>
<td>Linear Potentiometers</td>
<td>5</td>
<td>half</td>
<td>1</td>
<td>2</td>
<td>0.1%</td>
</tr>
<tr>
<td>Clip Gauges</td>
<td>4</td>
<td>full</td>
<td>-</td>
<td>1,000</td>
<td>200 dlu/mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-</td>
<td>500</td>
<td>100 dlu/mm</td>
</tr>
<tr>
<td>Strain Gauges</td>
<td>4</td>
<td>quarter</td>
<td>50</td>
<td>100</td>
<td>0.085 dlu/µe</td>
</tr>
</tbody>
</table>

(1) Resolution of Linear Potentiometer depends on its maximum travel
(2) dlu : data logger units

3.9 COMPONENTS OF INTERSTOREY DISPLACEMENT

3.9.1 General

One important aspect of the experimental programme was the determination of the sources of the lateral displacement of the columns of the test units.
The instrumentation to enable the calculation of the different sources of displacement was concentrated in those regions of the test units where large inelastic displacements were expected. For the units tested the instrumentation was concentrated in the beam spans where plastic hinges were expected to develop, and in the beam-column joint regions to measure shear distortions. However, column deformations were only partially measured or not measured at all. The sections below describe the procedures used to find these sources of displacement.

### 3.9.2 Column Deformations

The column of the units were designed to remain in the elastic range. Hence, no major contributions to the total horizontal displacements of the units were expected to arise from deformations within the columns themselves. Except in the initial elastic load cycles where the flexibility of the columns could make an important contribution to the horizontal displacement imposed.

Several approximate methods can be used to evaluate the deformations due to the columns. One method is to assume an approximate value for the elastic properties of the cracked section of the member. Another method is to estimate the column deflection from a moment curvature analysis [Park and Paulay (1975)]. In the latter method the calculated deflection due to flexure may be larger than the actual deflection measured if the effect of tension stiffening has been disregarded. In both cases, approximations for the shear displacements and the contribution of the fixed-end rotation need to be added.

The moment-curvature approach was selected for the evaluation of the deformation of the columns of Units 1 to 4. The curvature diagram was assumed to be proportional to the bending moment diagram. This assumption is not necessarily exact since the curvature in the columns may be affected by the tension shift effect [Park and Paulay (1975)]. Also, a strain penetration of 15\(d_{n}\), where \(d_{n}\) is the nominal diameter of the longitudinal column bar anchored, was assumed to estimate the fixed-end rotation or in other words the additional rotation at the beam ends due to strain penetration in the beam-column joint core. Shear deformations and the effect of tension stiffening were ignored.

In Units 5 and 6, the deformation of the columns was partly determined from measured displacements at the ends of the columns. In addition, a calculated deflection, as for the column of Units 1 to 4, was added to account for the deformations outside the instrumented region.

The instrumentation placed in the columns of Units 5 and 6 at the beam faces was aimed to measure the fixed-end rotation as well as the part of the flexural deformations within the column. Also, DEMEC gauges monitored the shear displacement across the horizontal construction joints. The arrangement of this instrumentation is illustrated in Fig. 3.29.

### 3.9.3 Beam-Column Joint Shear Distortion

Diagonal readings in the beam-column joint panel enabled the average shear distortion to be estimated. From Fig. 3.33 it can be demonstrated that the joint shear strain, \(\gamma\), is given by
Fig. 3.33 - Shear Distortion in a Beam-Column Joint Panel.

Fig. 3.34 - Evaluation of the Fixed-End Rotation in a Beam at the Column Face.
\[ \gamma = \frac{(\delta_s - \delta_s')}{2\delta_j} \left( \tan \eta + \frac{1}{\tan \eta} \right) \]  

(3.7)

where \( \delta_s \) and \( \delta_s' \) are the changes in length of the diagonals, \( \ell_j \) is the initial length of the diagonals and \( \eta \) is the angle of the diagonals to the horizontal.

3.9.4 Beam Deformations

3.9.4.1 Fixed-End Rotation

The fixed-end rotation in a beam is caused by the elastic and inelastic deformations of the longitudinal bars anchored in the joint core or by global slippage of these bars or by both. The fixed-end rotation can be directly evaluated from the pull out of the longitudinal bars. An approximate method is to assume that the fixed-end rotation is given by the pair of linear potentiometers located in the beam next to the column faces, as shown in Fig. 3.24. This method has the disadvantage that the fixed-end rotation so obtained is overestimated due to the strain penetration of the longitudinal bars in the beam span. This approximate method was adopted in this study. From Fig. 3.34 the fixed-end rotation, \( \theta_{fe} \), and the corresponding vertical beam displacement, \( \delta_{b,fe} \), evaluated at a distance \( x \) from the column face can be obtained as

\[ \theta_{fe} = \frac{\delta_s - \delta_s'}{h_p} \]  

(3.8)

\[ \delta_{b,fe} = \theta_{fe} x \]  

(3.9)

where \( h_p \) is the distance between the linear potentiometers.

3.9.4.2 Flexural Deformations

In this study, flexural deformations, \( \delta_{b,ff} \), are defined as those estimated from the discrete rotations of each segment of the beam span, obtained from a pair of linear potentiometers located at the top and bottom chords. From Fig. 3.35

\[ \delta_{b,ff} = \Sigma \left( \frac{\delta_s - \delta_s'}{h_p} \right) (x - x_p) \]  

(3.10)

Eq. 3.10 assumes the Navier-Bernoulli hypothesis of plane sections remain plane after deformation to calculate the flexural component. Other component of the beam deformation is allocated to shear, as discussed in the following section.

3.9.4.3 Beam Shear Deformations

Celebi and Penzien (1973) and Ma et al (1976) and have pointed out that shear deformations in the span of a short beam are coupled with flexural deformations. Hiraishi (1984) has also shown that in structural walls a similar effect occurs. Shear deformations obtained using Eq. 3.7 lead to errors because the change in length of the diagonals is also affected by the extension of the tension chord due to flexure in the beam. Fig. 3.36 shows a case of an apparent shear deformation due to the effect of
Fig. 3.35 - Evaluation of the Beam Flexural Displacements.

Column face gauged with top & bottom linear potentiometers

Fig. 3.36 - Apparent Shear Displacement Due to Elongation of a Diagonal Gauge.

\[ \frac{(\delta_s - \delta_s')}{h_p} (x - x_p) \]

Apparent Shear Displacement:
\[ \frac{\ell'_s - \ell_s}{2\sin\eta} \]
the fixed-end rotation. In this study another methodology was developed to assess the shear deformations in the plastic hinge regions of the beams. Shear deformations elsewhere in the span of the beam were disregarded in spite of diagonal measurements taken. The different approach is discussed in the paragraphs below.

(a) Shear-flexure Action

Fig. 3.37 shows the kinematics of the plastic hinge of a beam where the flexure shear cracks have been lumped in an equivalent crack radiating from the centroid of the compression steel at the column face. At a distance \( x \) from the column face, the extension of the chords and the beam displacement due to flexure and fixed-end rotation, \( \delta_f \), are all known.

The shear component due to the diagonal shear-flexure cracking can be determined by assuming that the length of the compression strut, \( e_{cr} \), remains unchanged. From geometry this shear deformation, \( \delta_{s,cr} \), is given by

\[
\delta_{s,cr} = \frac{\delta_s}{\tan \eta} - \delta_f
\]  

(3.11)

The critical distance, \( x_{cr} \), at which the diagonal strut develops can be approximated by

\[
x_{cr} = \frac{V_s}{\frac{A_s}{s} f_{yt}}
\]  

(3.12)

where \( V_s \) is the theoretical shear in the beam associated with the lateral capacity \( H_s \), \( A_s \) is the area of transverse steel spaced at a distance \( s \) and \( f_{yt} \) is the measured yield strength of the transverse reinforcement. This expression assumes that the critical crack will cross the necessary amount of transverse reinforcement that is required to transfer the whole shear in the beam.

For practical purposes the value of \( x_{cr} \) taken to evaluate the shear deformation due to flexure can be approximated as to the nearest end of the segment covered by linear potentiometers.

(b) Sliding Shear

Fig. 3.38 illustrates another mode of shear displacement, termed here as sliding shear. Because of yielding of the stirrups, disintegration of the diagonal strut or because of relative sliding between cracks in the plastic hinge region, the length of the strut, \( e_{cr} \), will decrease and induce an additional vertical displacement, \( \delta_{s,cr} \), due to the sliding action. If the new length, \( e'_{cr} \), is known then the sliding shear displacement can be estimated as
Fig. 3.37 - Shear Displacement Due to Shear-Flexure.

\[
\delta_{b, sf} = \frac{\delta_s}{\tan \eta} - \delta_f
\]

Fig. 3.38 - Shear Displacement Due to Sliding Shear.

\[
\delta_{b, ss} = \frac{l_{st} - l'_{st}}{\sin \eta}
\]
In the test of Units 1, 2 and 3 the change in length of the diagonal strut relative to the column face was not measured by the diagonal gauges because of the space required by the DEMEC gauges (see Fig. 3.28 (a) and (b)). Nevertheless, photographic records were used to measure this change within ±3mm.

### 3.9.5 Member Deformations in Terms of Column Displacement

Having calculated the local member displacements or deformations it is necessary to transform them into horizontal column displacements. This procedure enables components of displacement to be determined as a percentage of the total displacement and a comparison to be made of the calculated displacement with that imposed during the test.

Figs. 3.39 and 3.40 show the main source of displacements considered in the test specimens. Displacements occurring in the beams or in the beam-column joint region can be transformed to an equivalent interstorey or column displacement as:

\[ \delta_{c,j} = \frac{\xi}{\ell_b} \delta_{b,j} \]

\[ \delta_{c,fe} = \frac{\xi}{\ell_b} \delta_{b,fe} \]  

\[ \delta_{c,ft} = \frac{\xi}{\ell_b} \delta_{b,ft} \]

\[ \delta_{c,a} = \frac{\xi}{\ell_b} (\delta_{b,af} + \delta_{b,am}) \]

where \( \delta_{b,j} \), \( \delta_{b,fe} \), \( \delta_{b,ft} \) and \( \delta_{c,a} \) are the equivalent column displacements due to the shear distortion at the beam-column joint region, the fixed-end rotation in the beams, the flexural deformations in the beams and the shear displacements occurring in the beams near the column face respectively, \( \ell_c \) is the distance between the column pins and \( \ell_b \) is the horizontal distance between column centrelines in Units 1 to 4 or between beam pins in Units 5 and 6.

The total column displacement, \( \delta_c \), calculated from the deformations components is:
Fig. 3.39 - Mode of Deformation of the H-Shaped Units 1, 2, 3 and 4.
Fig. 3.40 - Modes of Deformation of Cruciform Units 5 and 6.
\[ \delta_{co} = \delta_{c,e} + \delta_{c,j} + \delta_{c,fe} + \delta_{c,ft} + \delta_{c,\theta} \]  

(3.15)

where \( \delta_{c,e} \) is the estimated displacement from deformations within the columns.

With regard to Fig. 3.39, the angles \( \alpha_1 \) to \( \alpha_3 \) of the slaved column to the vertical do not need to be zero. In theory, however, the sum of these angles should be zero, providing that the displacement occurring in both columns is the same.

3.10 CONCLUSIONS

1. This chapter describes the construction and testing of the six beam-column subassemblages incorporating precast concrete. The test programme was divided into two parts. The first series of tests was conducted to evaluate the cyclic load performance of different connection details located the midspan region of beams. The second series of tests was conducted to evaluate the performance of different connection details located at the beam-column joint.

2. Test results on high strength low viscosity grouts indicated that high compressive strengths are easily attainable. However, one of the main factors in the use of these type of grouts is to ensure the fluidness avoiding excessive segregation and bleeding. Tests on the mechanical properties showed that the ratio of the splitting strength to the compressive strength and the enhancement of the compressive strength due to a lateral confining pressure is similar to that to concrete. However, the Young’s modulus is significantly lower.

3. An alternative to the grouting procedure conducted in this programme could be the use of pre-packed aggregates inside the metallic ducting in which a water-cement grout is easily pumped through an orifice in the bottom joint between the lower column and the precast member.

4. The welding of reinforcing bars to steel plates showed an ineffective underfill region under the bars. It is recommended that, in lieu of more data, the welding commence at the midheight of the reinforcing bar be at a slope not greater than 60°.
4.1 INTRODUCTION

This chapter presents test results from the three H-shaped conventionally reinforced precast concrete units, connected at the midspan of the beam, with short member sizes typical of moment resisting perimeter frames. The main aim of this series of tests was to observe the behaviour of different short connection details at the midspan of the beam and the effect of their proximity to the critical regions where plastic hinges were expected to develop. Units 1, 2 and 3 were designed to induce in the beam the maximum allowed shear stress \( u^0 = \frac{V^0}{bd} = 0.30 \sqrt{f_c} \) permitted by the Concrete Design Code [NZS 3101 (1982)] when plastic hinges are expected to form in conventionally reinforced beams with symmetrical reinforcement.

Complete reinforcing details and the test regime were presented in Sections 3.2.2.1 and 3.7 respectively. Test results are presented in terms of the load run number (see the run numbers marked on Figs. 4.2 and 4.3) instead of the displacement ductility factor. The mapping between these systems can be found in Fig. 3.25 where the load sequence was shown.

4.2 UNIT 1

4.2.1 General Behaviour

The first unit to be tested, Unit 1, was designed following an existing connection detail of a building already constructed. The longitudinal beam reinforcement was overlapped at midspan using 180° hooks and transverse rods in contact with the concave side of the hooks to improve the anchorage conditions (see Fig. 3.17). The splice of the bars commenced at a distance of 1.46d from the column faces, where d is effective depth of the beam.

The test took four weeks to complete and attained load cycles in the inelastic range up to a displacement ductility factor of \( \mu_\Delta = 5.36 \), at an interstorey drift of 2.3%, before losing its lateral load capacity by more than 20% of the maximum recorded. The cumulative displacement ductility factor attained by the test unit was \( \sum \mu_\Delta = 59 \) which leads to an available displacement ductility factor of \( \mu_a = 7.4 \) according to the method discussed in Section 3.7 to establish the available ductility of test subassemblages. The overall performance of the test and the connection at midspan was satisfactory in terms of strength and available ductility. However, severe pinching of the hysteresis loops occurred during the final stages of the test.
In the initial cycles in the elastic range, a small error occurred because the unit was loaded to 75% of the ideal lateral load capacity calculated using the nominal properties of the materials instead of the actual lateral load capacity, \( H_\text{e} \), estimated from the measured properties. In terms of \( H_\text{e} \), the lateral load imposed was 67% at the peak of the load controlled runs 1 to 4. Therefore the displacement history applied to the unit was 67/75 of that if this error had not occurred, that is, the actual displacement ductility factors applied to the unit are smaller than the nominal displacement ductility factors intended to be initially imposed. It is believed that this small error did not bear any significance in the test results.

Other sources of error during the test occurred with the instrumentation used to determine the net lateral displacement of the columns of the unit. Secondary stands required for fixing the linear potentiometers A, B, C and C' in Fig. 3.26 were fixed to the test floor. Unfortunately the forces applied to the edge of the floor during the tests were of such magnitude that the floor itself flexed upwards under the column in tension, causing an extra rigid body displacement that was not recorded by the displacement transducers. Independent measurements taken with a theodolite and a high precision level at the peak of each load run confirmed these movements. The unrecorded rigid body movement resulted in an extra lateral displacement in the load controlled cycles in the elastic range that affected the magnitude of the first yield displacement. A further problem that developed in the test of this unit was caused by the high friction forces in the free-to-slide mechanism of the test frame that had been designed to permit for the expected elongation of the beam. As a result, some axial load had to be transmitted through the beam enhancing its flexural capacity and hence the lateral load capacity of the unit itself. These friction forces caused a redistribution of the forces applied by the hydraulic actuators.

Fig. 4.1 illustrates visible cracking of the beam of Unit 1 at different stages during the test. Some very fine randomly arranged cracks developed in the sides of the beam in the cast in place concrete joint at midspan before the commencement of the test. Their pattern suggest they were caused by shrinkage of the concrete.

In the cycles in the elastic range, load runs 1 to 4, several cracks formed regularly in the end of the beams extending to about 600mm from the columns faces. The cracks commencing in the top generally extended further than the cracks commencing in the bottom of the beam. Two main cracks propagated through the web of the beam. The first one was a crack at the interface between the beam and the column and the other one was a diagonal crack appearing between 400 and 600mm from the column and heading towards the compression region in the beam at the column face. Cracks in the beam that opened in one load run nearly closed in the next. In the second in the elastic range, only propagation of the existing cracks was noticed. Cracks in the beam-column joint regions opened only in the loading runs where the column was subjected to axial load in tension and closed in the opposite runs.

In load runs 5 to 8 to \( \mu = \pm 1.78 \) the beams showed signs of the initial yielding of the longitudinal reinforcement. Many cracks propagated in the web of the beam near the column faces with a typical fanned pattern. The main cracks at this stage were observed in the beam at the column face where crack widths of 2.5 and 3.5mm were observed. Previously open cracks at the level of the beam longitudinal
Fig. 4.1 - Cracking of Beam of Unit 1 at Different Stages During Testing.

(a) At $\mu_\Delta = -3.57 x 1$

(b) At $\mu_\Delta = -5.36 x 1$

(c) At $\mu_\Delta = -5.36 x 4$
reinforcement in tension did not close in the opposite load run. Hence lengthening of the beam occurred. Diagonal cracks in the web of the beam commenced to show an increase in their width and a maximum crack width of 1.0mm at the beam mid-height was observed. Cracks elsewhere in the test unit remained very small suggesting that plastic hinges were developing at the beam ends as expected.

The plastic hinges in the beam ends spread in the loading cycles to $\mu = \pm 3.57$, load runs 9 to 12 (see Fig. 4.1 (a)). Several diagonal cracks extended and widened, especially those near the column faces where crack widths at the mid-height of the beam of 2mm were recorded. Also, initial spalling of the concrete in the web of the beam was observed in these load runs. Spalling of the concrete resulted because of the grinding action caused by the relative shear displacement along cracks, especially in the south end of the beam where cracks formed in opposite load runs had interconnected. Shear deformations in the plastic hinge regions were evident at this stage. The crack widths in the beam at the column faces grew to 6mm.

The final loading cycles to $\mu = \pm 5.36$, load runs 13 to 20, were characterized by the grinding action between the cracks in the plastic hinge regions of the beams because of the sliding shear deformations taking place in these regions (see Fig. 4.1 (a) and (b)). The growth in height of the beam became evident, indicating that the stirrups had been subjected to rather large tensile strains beyond yield. Horizontal cracks at the level of the longitudinal reinforcement in the beam extended to the cast in place joint at midspan and were usually initiated in the spaces provided for the targets to measure the reinforcement strains. These cracks and the shear deformation observed suggest that dowel action became an important mechanism for transferring the shear. The cracks in the beam at the column faces did no grow extensively during these load runs. However, crack widths in the diagonal cracks became rather large with values up to 5mm recorded at the mid-depth of the beam. It was also seen that the linear potentiometers in the top and bottom chords of the beam located in the plastic hinges tilted due to kinking of the steel rods embedded in the diagonally cracked concrete. Large shear displacements were also noticed since the tip of the linear potentiometers moved vertically a considerable amount. In load run 17 a hairline crack crossed one corner of the cast in place concrete at the midspan connection. This crack did not propagate nor increase its width in the subsequent loading cycle.

4.2.2 Load Displacement Response

The lateral load versus lateral displacement response of Unit 1 during the loading cycles in the elastic range, load runs 1 to 4, is illustrated in Fig. 4.2. The second loading cycle resulted in little energy being dissipated since most of the energy dissipated in the first cycle was caused by the cracking of the test unit whereas the second cycle was characterized by the extension of the existing cracks. A loss of stiffness was also observed in the second cycle perhaps caused by some of the cracks in the beam that did not fully close. The average of the measured "elastic" stiffness was only 34% of that theoretically derived in Section 3.2.1, where only one-half of the gross section properties were used for the beam and the columns to estimate the initial stiffness allowing for initial cracking. This measured value of stiffness was probably underestimated, because of the rigid body rotation of the beam which was only partly measured. Nonetheless, it is believed that the unrecorded rigid body rotation would not account for the large difference. This difference is instead largely due to the strain
Fig. 4.2 - Lateral Load-Lateral Displacement Response of Unit 1 During Loading Cycles in the Elastic Range.

Fig. 4.3 - Lateral Load-Lateral Displacement Response of Unit 1.
penetration of the longitudinal reinforcement in the beam and the column into the beam-column region, which causes a discrete fixed-end rotation at the interfaces of the beam-column joints, to the curvature distribution in the short beam and columns which does not precisely follow the bending moment in the member because of the influence of the tension shift effect caused by diagonal tension cracking, and, to a lesser extent, to the flexibility of the beam-column joints.

Fig. 4.3 shows the complete lateral load versus lateral displacement response of Unit 1. The post-elastic slope of the envelope of the hysteretic response, estimated as the slope of the line passing between the theoretical point at first yield and the point at the maximum load in runs 13 and 14, was 7.8 and 6.2% of the initial "elastic" slope respectively. The measured energy dissipated in each cycle as well the cumulative energy absorbed was normalized using the energy dissipated by an ideal structure responding bi-linearly and having the same elastic and average post-elastic slopes.

In the first post-elastic load run, load run 5 to \( \mu_\Delta = +1.78 \) the measured lateral load exceeded the theoretical lateral theoretical load, \( H_a \) by 11.5%. The measured lateral load in the reverse load run 6 was slightly smaller. This trend was observed during the whole experiment and is believed to be linked to softening of the unit in the previous load run. In load runs 7 and 8 to the same displacement factor the lateral load attained was slightly below that attained in the first cycle. The energy dissipated in these cycles was only 68 and 40%, respectively, of that of the ideal bi-linear loops. The lower energy dissipated in the second cycle was probably caused by the closing of the previous diagonal cracking as well as by the smaller of extension of cracks that was observed.

In the subsequent load runs 9 to 12 to \( \mu_\Delta = \pm 3.57 \) the measured lateral load increased up to \( 1.19H_a \) and only a small drop in the measured load was recorded in the second loop to the same displacement ductility factor. The shape of the hysteresis loops became pinched, implying a loss in energy dissipation capacity caused by the closing of the diagonal cracks in the plastic hinge regions of the beam. The energy dissipated in these cycles was 63 and 42% of that of the ideal bi-linear loops.

The maximum overstrengths of \( 1.35H_a \) and \( 1.28H_a \) were recorded during load runs 13 and 14 to \( \mu_\Delta = \pm 5.76 \). In terms of the nominal shear stresses the maximum overstrength recorded in the beam plastic hinges was \( \sigma^* = 0.37\sqrt{f_c} \). At this stage it became obvious that the friction developed in the free-to-slide mechanism at the support at the bottom of the south column was causing an enhancement of the flexural strength of the beam, which combined with strain hardening of the longitudinal reinforcement in the plastic hinges accounted for the relatively high measured overstrengths. The subsequent load runs 15 to 20 showed a gradual deterioration of the stiffness and energy dissipation capacity. In fact, the energy absorbed in these cycles was only 46, 27, 19, 15% of the ideal bi-linear loops. This considerable loss of stiffness and energy dissipation was associated with the gradual reduction in length due to grinding of the diagonal strut carrying most of the shear in the plastic hinge regions of the beam. Dowel action, then, became an important mechanism for transferring the shear in the plastic hinge regions. By observing the shape of the hysteresis loops in Fig. 4.3, it can be estimated that approximately 20 to 30% of the beam shear in load runs 13 to 20 could have been carried by this mechanism. A loss of lateral load capacity of 20% of the maximum capacity was recorded in run 19 accompanied by very significant stiffness degradation. However it is believed that had the unit
been displaced further it would probably had attained a lateral load capacity comparable to the maximum recorded. Nonetheless, the test was stopped at this stage because the redistribution of forces implied by the friction at the free-to slide pin caused the loading frame to reach its service load capacity at one end.

The cumulative post-elastic energy dissipated by the test unit up to load run 20 was only 33% of the ideal cumulative energy of the ideal bi-linear loops.

### 4.2.3 Decomposition of Lateral Displacements

The components of the lateral displacement at the peak of each load run are shown in Fig. 4.4. These components were estimated following the method described in Section 3.9. Calculated displacements in Fig. 4.4 are shown as a percentage of the measured lateral displacement. The top part of this figure displays the results of the load runs to a positive displacement while the bottom part shows the components of the load runs to a negative displacement.

![Fig. 4.4 - Components of Lateral Displacement of Unit 1 at Peaks of Load Runs.](image)
In the cycles in the elastic range, the total estimated lateral displacement was on average 74% of that measured. The difference is probably due to the underestimation of the rigid body rotation of the test unit leading to an overestimate of the lateral displacements.

The fixed-end rotation of the beam, due mainly to strain penetration of longitudinal bars (see Section 4.2.1), was the major source of displacement in the initial load runs and its contribution remained important throughout the test, especially in the first cycle to each particular displacement ductility factor. The flexural deformations of the beam had a major contribution to the total displacement of the test unit in the cycles in the elastic range but their contribution to the total displacement gradually diminished as the test progressed. However, the shear displacements in the beam made a small contribution initially, even smaller than what an elastic analysis would indicate, but became the dominant contribution to the total deformation in the final loading cycles of the test. The large shear deformations in the plastic hinge regions of the beam were also associated with the longitudinal extension of these regions, since cracks that were open in a previous load run did not close in the next, increasing the tendency for sliding shear action.

4.2.4 Beam Curvature and Rotational Ductility Factors

Figs. 4.5 and 4.6 show the measured curvature and rotational ductility factors of the beam of Unit 1 estimated from the second set of linear potentiometers placed in the beam chords, commencing at 25mm away from the column faces, where the effect of diagonal cracking and the crack at the interface would be less important. The gauge length for calculating the curvature was 145mm. Both, the curvature and rotation at first yield were extrapolated using the average of the estimated curvatures and rotations at the north and south ends of the beam and following the same method outlined in Section 3.7 for determining the displacement ductility factors. The calculated curvature at first yield, \( \phi_y \), was 0.0027 radians/m and the rotation at first yield, \( \theta_y \), was 0.0033 radians. The estimated curvature ductility factors, \( \mu_\phi \), in load runs 6 to 8 were rather small because yield of the reinforcement was mainly confined to the first segments of the beam next to the column faces. Beyond these load runs, the estimated curvature ductility factors show no definite trends. One reason is that the estimated curvatures are very sensitive to the position and width of the cracks forming in the concrete at and around the gauged region. In addition, at the end of the test it would be expected that some errors could be induced in the readings from the linear potentiometers caused by the large shear distortion at each end of the beam.

The rotational ductility factors, \( \mu_\theta \), shown in Fig. 4.6 include the measured rotations of the north and south ends of the beam and the fixed-end rotations at the face of the columns. Also presented in Fig. 4.6 are the predicted rotational ductility factors estimated from a plastic analysis assuming that all deformations were due to rotations at the beam ends. The predicted values always overestimate those measured by a rather large margin. There are two main reasons for the disagreement observed. The first and main reason is the difference between the estimated net lateral displacement and the actual one. A second reason is the shear deformations which occurred in the second cycles to a displacement ductility factor meant that not so much flexural rotation was required of the beam. This effect is more
Fig. 4.5 - Beam Curvature Ductility Factors of Unit 1.

Fig. 4.6 - Beam Rotational Ductility Factors of Unit 1.
pronounced in the cycles near the end of the test when the shear deformations were the main source of lateral deformation.

4.2.5 Beam Longitudinal Bar Strains and Forces

Average strains of the east and west beam longitudinal bars of the beams are shown in Fig. 4.7. These strains were measured using a manual DEMEC gauge with a gauge length of 102mm. In addition, Fig. 4.8 illustrates member strains at the level of the longitudinal reinforcement estimated from the top and bottom linear potentiometers using the procedure discussed in Section 3.8.3.2. Both figures show the gradual increase of tensile strains in the plastic hinge regions at the beam ends. Both measuring systems show similar trends although the member strains near the beam ends in Fig. 4.8 are more erratic, perhaps due to the main cracks passing through the orifices in the cover concrete of the beam provided around the steel rods present for fixing the linear potentiometers. It is important to stress that the differences between the strains in Figs. 4.7 and 4.8 will depend on the quality of the bond between the reinforcing bars and the concrete, which will deteriorate in the plastic hinge regions of the beams as the test progresses.

From load runs 9 to 10 to $P_c = \pm 4$ large residual tensile strains were detected in the beam in the plastic hinge regions up to a distance of 335mm from the column faces. The residual tensile strains in the plastic hinges led to a cumulative horizontal lengthening of the beam.

Fig. 4.9 displays the forces in the top and bottom longitudinal reinforcement of the beam of Unit 1 calculated from the measured bar strains shown in Fig. 4.7. The following procedure was carried out to estimate these forces:

a) Estimate the east and west bar stresses from measured data using the steel stress-strain model discussed in Chapter 2 and the mechanical properties of the reinforcement presented in Table 3.8.

b) Average east and west bar stresses.

c) Estimate the force in each section by multiplying the average stress by the nominal steel area.

The extent of yielding shown in Fig. 4.7 is apparently in disagreement with that shown in Fig. 4.9, especially with regard to the top bars in the north side of the beam. In Fig. 4.7 it appears that yielding spread over a distance $d$ from the north column face, where $d$ is the effective depth of the beam, while Fig. 4.9 shows a more limited length of yielding. The reason for this apparent discrepancy is that the kinked east bar yielded a significant amount while the straight west bar remained elastic. The east bars were kinked to create the offset required for overlapping the bars in the connection region at midspan. Their average strain then shows a value larger than the yield strain. However, the average stress shows a value below yield. At these sections in general, the measured strains on east and west
Fig. 4.7 - Average Beam Longitudinal Bar Strains of Unit 1.
Fig. 4.8 - Member Strains at the Level of Longitudinal Reinforcement of Unit 1.
Fig. 4.9 - Longitudinal Bar Forces in the Beam of Unit 1.
bars exhibited rather different strain levels with the kinked bar consistently showing larger tensile strains, beyond yield in the cycles in the inelastic range, than the straight bar.

In the elastic cycles the forces in the top bars were larger than in the bottom bars, as shown in load runs 1 and 2 in Fig. 4.9. A reason for this behaviour is that there was a different moment demand, because for the same applied lateral displacement the lateral load in the column in compression was larger than that for the column in tension.

With the exception of the bottom bars when the test unit was subjected to positive displacements, the plastic hinge regions in the beams were confined to within a distance d from the column face. The tension shift at the beam ends was, on average, approximately equal to d, but rapidly decreased to almost zero at the midspan of the beam.

The forces in the compression bars near the beam ends were initially very small, implying that the concrete was transferring most of the compression, but they increased to approximately two thirds of the force in the tension bars in the final cycles of the test.

4.2.6 Longitudinal Bar Stresses at Midspan

Fig. 4.10 illustrates the stress profile of the beam longitudinal reinforcement in the connection region. The most remarkable feature observed was that there is no lapping action between the overlapping bars. The tension bars are acting as if they were anchored in this region.

The maximum relative longitudinal movement of the bars, measured also with DEMEC gauges, was only 0.1mm. This shows the adequate anchorage conditions of the bars. The transverse rods played an important role in distributing the bearing stresses of the bar in tension and avoiding splitting of the concrete due to the radial nature of the stresses around the longitudinal bars.

4.2.7 Stresses in the Transverse Reinforcement at Midspan

The transverse reinforcement in the connection region at the midspan of the beam was instrumented with 8 electrical resistance foil strain gauges. Recorded strains indicate that the level of the stresses in the vertical and horizontal sides of the stirrups was very low, of the order of 0.1f. This level of stresses agrees with the observed behaviour of the cast in place joint where no significant cracks were detected.

4.2.8 Elongation of the Beam

The measured total elongation of the beam span of Unit 1 plotted against the measured lateral load is illustrated in Fig. 4.11. A small residual elongation was recorded at the end of the cycles in the elastic range, implying that some cracks did not fully close. A feature of the elongation of the beam is that most of it occurred in the first cycle to a new value of displacement ductility factor when the flexural capacity of the test unit was attained. Also, some elastic recovery was observed when
Fig. 4.10 - Bar Stresses at the Midspan of the Beam of Unit 1.
unloading upon reversal. The elongation accumulated in the load runs because of the residual tensile strains in the top and bottom longitudinal bars. No important elongations were recorded in the third and fourth cycles to $\mu_a = \pm 5.36$ since the unit did not develop its flexural capacity in those cycles. The maximum measured total elongation was 26mm recorded at the peak of load run 19 to $\mu_a = 5.36$.

4.3 UNIT 2

4.3.1 General Behaviour

The midspan connection detail used in Unit 2 consisted of 90° double hooked "drop in" bars which lapped two thirds of the longitudinal beam reinforcement. Transverse rods were placed in contact with the concave side of the hooks to improve the bearing conditions (see Fig. 3.18). The connection at the midspan of the beam commenced at a distance of 1.23d from the column faces.

The free-to-slide mechanism at the support at the end of the south column was modified to reduce the friction forces which had developed during the first test (see Section 3.3). Also the instrumentation used to determine the net lateral displacement of the test unit was attached to steel frames fixed outside the test floor area, to avoid the errors due to the flexibility of the concrete floor.

The test of Unit 2 was conducted over a period of two weeks. A decrease of more than 20% of the maximum measured lateral load occurred in fourth cycle to $\mu_a = -6$ at a drift of 2.1%, due to stiffness degradation. The cumulative displacement ductility factor attained before failure was $\Sigma \mu_a = 60$ equivalent to an available displacement ductility factor of $\mu_a = 7.5$ (see Section 3.7). Further cycles to $\mu_a = \pm 8$ showed some reserve of strength since the lateral load measured in this cycle was comparable
with the maximum lateral load measured in the test. Shear deformations in the beam plastic hinge regions caused severe pinching of the measured hysteresis loops, especially near the end of the test. The overall performance of the connection detail was very satisfactory. Also, its relatively close proximity to the critical regions of the beam apparently did not influence the overall response of the test unit.

The visible cracking of the beam of Unit 2 at different stages during the test is shown in Fig. 4.12. In the cycles in the elastic range, the cracks spread in the beam from the column faces to the cold joint between the precast concrete members and the cast in place concrete at midspan. The main cracks in these load runs were the vertical cracks at the column faces, which opened as a result of the strain penetration of the beam longitudinal bars into the beam-column joint region. Cracks at the cold joints near midspan propagated beyond the mid-depth of the beam but they were rather fine. Cracks also appeared in load run 3 around the 90° hook in the top north and bottom south bars in the connection region at midspan. The width of these cracks was only 0.02mm.

In load runs 5 to 8 to $\mu_k = \pm 2$, many cracks extended and widened in the expected plastic hinge regions at the beam ends. The cracks at the faces of the columns were 2.5 to 3mm wide at the tension side. Although their width decreased towards the compression side, they remained open over the full depth of the beam. The maximum width of the diagonal shear-flexure cracks at the mid-depth of the beam was 0.3mm. The vertical cracks at the cold joint between the precast members and the cast in place concrete opened to a width of 0.6mm at the mid-depth of the beam. Diagonal cracks commenced from these cracks and headed towards the compression region in the beam at the column faces. The cracks around the hooked bars also widened and extended. However their maximum width was only 0.2mm.

In the cycles to $\mu_k = \pm 4$, load runs 9 to 12, some diagonal cracks in the plastic hinge regions of the beam became rather wide (see Fig. 4.12 (b)). Crack widths in this region, measured at the mid-depth of the beam, were as large as 3mm. The width of the cracks suggested that plastic hinge regions had commenced to extend from the column faces and that yielding of the stirrups in the beam ends was likely to have occurred. In addition, relative shear displacements of 1mm between the sides of the cracks were observed. The cracks at the cold joint in the midspan region also widened, as if the reinforcing steel in this region had yielded. Relative shear displacements of 1mm along these cracks were observed. In load run 9 two splitting cracks running parallel to the reinforcing bars protruding from the precast members were observed in the cast in place concrete at the bottom south side of the beam. In load runs 11 and 12 several small diagonal cracks formed in the web of the beam in the cast in place concrete.

The vertical elongation of the beams in the plastic hinge regions became evident in the first cycle to $\mu_k = \pm 6$ when the measured expansion at a distance of 280mm from the column faces was 11 and 8mm at the north and south ends, respectively. Spalling of the concrete at the beam ends was also observed at this stage due to the grinding action caused by the sliding shear displacements along the cracks. Many fine cracks formed in the cast in place concrete during these loading cycles, as is seen in Fig. 4.12 (c). The height of the beam in the plastic hinge regions increased with further cycles to
(a) At $\mu_\Delta = -4x1$

(b) At $\mu_\Delta = -6x1$

(c) At $\mu_\Delta = -8x1$

Fig. 4.12 - Cracking of Beam of Unit 2 at Different Stages During Testing.
the same displacement ductility factor due to yielding of the stirrups. The stiffness of the unit degraded to a point that the measured lateral load in run 19 was below the 80% of the maximum previously attained load. In load run 19 the increase in height of the beams in the plastic hinge regions was 20 and 12mm in the north and south end, respectively. In this load run the relative shear displacement along the cold joint was 2mm and the maximum width of the cracks in the cast in place region at midspan had increased to 0.5mm.

The test unit was taken to $\mu_\delta = \pm 8$ in load runs 20 and 21 to observe on whether the capacity of the unit had been reduced. In this cycle the plastic hinge regions of the beam appeared severely damaged, but they were still able to transfer shear forces of similar magnitude to the maximum ones recorded in the first loading cycle to $\mu_\delta = \pm 6$.

### 4.3.2 Load Displacement Response

Fig. 4.13 shows the lateral load versus the lateral displacement of Unit 2 in the cycles in the elastic range. In the first cycle more energy was dissipated that in the second cycle, since most of the cracking took place in load runs 1 and 2. The measured "elastic" stiffness of Unit 2 was only 42% of the stiffness predicted in Section 3.2.1. The causes for this large difference were the same of Unit 1 (see Section 4.2.2). The interstorey drift at the experimental first yield displacement was 0.35%, and was 0.31% at the theoretical dependable lateral load capacity of the Unit.

Fig. 4.14 illustrates the complete lateral load versus lateral displacement response of Unit 2. The post-elastic stiffnesses estimated as the slopes of the lines passing through the point at first yield and the point at maximum overstrength in runs 13 and 14 were 3.9 and 2.5% of the "elastic" stiffness for the positive and negative loading runs, respectively. An average stiffness of 3.2% of the "elastic" stiffness was used for defining the properties of the ideal bi-linear loops used to normalize the energy dissipated in each cycle.

In load run 5 to $\mu_\delta = \pm 2$ the measured lateral load was 5% above the theoretical load $H_a$. In the reverse load run the lateral load did not reach the theoretical value but was only 4% below it. The lateral loads measured in the load runs to a negative displacement were always below those previously attained in the positive load runs, a phenomena that was consistent with the behaviour of the Unit 1. The second cycle to the same ductility factor showed very little strength degradation but the energy dissipated was much smaller, being 77% and only 47% of the ideal bi-linear loops for the first and second loading cycles, respectively.

The load runs 9 to 12 to $\mu_\delta = \pm 4$ showed the same trends as the previous cycles. However, pinching was more severe in those load runs due to the large diagonal cracks that had to be closed and, perhaps, due to some initial yielding of the stirrups in the plastic hinge regions at the beam ends. The energy dissipated in these cycles was 63 and 36% of the ideal bi-linear loops, respectively.

The maximum measured loads of $1.19H_a$ and $1.12H_a$ were attained in load runs 13 and 14 in the first cycle to $\mu_\delta = \pm 6$. The maximum measured load in terms of the nominal shear stress in the
Fig. 4.13 - Lateral Load-Lateral Displacement Response of Unit 2 During Loading Cycles in the Elastic Range.

Fig. 4.14 - Lateral Load-Lateral Displacement Response of Unit 2.
plastic hinges of the beam was $\psi = 0.31\sqrt{E}$. The measured hysteresis loops of the further loading cycles were severely pinched because of the large shear deformations in the plastic hinge regions of the beam, as can be seen in Fig. 4.14. The shear in the beam at low levels of lateral load was evidently carried by dowel action. By observing the shape of the loops it is believed that 20 to 30% of the beam shear was carried by this mechanism. Associated with the pinching of the loops was the degradation of the stiffness that caused the lateral load capacity in load run 19 to drop by more than 20% of the maximum measured load. The energy dissipated in these cycles was 44, 27, 21 and 15% of the ideal bi-linear loops, respectively. The maximum relative longitudinal displacement between the overlapped hooks was 1.4mm and was measured in load run 19.

The test unit was subjected to a final cycle to $\mu_a = \pm 8$ to observe whether the theoretical lateral load capacity was still attainable. In load runs 21 and 22 the measured lateral load was comparable with the maximum previously measured lateral load indicating that at this level of interstorey drift of 2.8% the unit had some reserve of strength.

The cumulative post-elastic energy dissipated by the test unit up to load run 20 was 33% of the energy dissipated by the ideal bi-linear loops, which was about the same as the energy dissipated by Unit 1.

4.3.3 Decomposition of Lateral Displacements

Depicted in Fig. 4.15 are the components of the lateral displacement at the peak of the load runs of the test of Unit 2. In the first stages of the test the fixed-end rotation of the beam, due mainly to strain penetration of the longitudinal bars, accounted for an important part of the lateral displacement imposed. Flexural displacements within the beam were also important. On the other hand, shear displacements in the beam plastic hinge regions did not account for a large percentage of the lateral displacement in the first stages of the test, but they became the dominant mode of deformation from load run 10 at the first cycle to $\mu_a = -4$.

4.3.4 Beam Curvature and Rotational Ductility Factors

The curvature ductility factors of the second set of linear potentiometers in the beam commencing at 10mm from the column faces are shown in Fig. 4.16. The gauge length covered by the displacement transducers was 170mm and, in theory, they should not include the effects of the strain penetration of the longitudinal beam bars. The calculated initial curvature at first yield, $\phi_y$, was equal to 0.0045 radians/m, a rather large value that was probably influenced by the interface crack at the beam ends. The curvature at first yield determined from the first set of strains measured in the longitudinal reinforcing steel was equal to 0.0027 radians/m, which appears to confirm the above hypothesis. Curvatures in this gauged length were erratic since the distribution of cracking was not uniform in the region. The first set of linear potentiometers was placed at 10mm away from the column face and hence the cracks in the beams at the column faces passed through the orifices where the steel rods protruded from the beam and shared their width between the first and second set of displacement transducers.
This effect is clearly seen in Fig. 4.16 in the positive displacement runs beyond load run 9 in the south end of the beam.

The rotational ductility factors are depicted in Fig. 4.17. The initial rotation at first yield including the fixed-end rotations, $\theta_y$, was found in a similar way to the curvature at first yield and its value was equal to 0.0036 radians. The rotation at the loading cycles to $\mu_4 = \pm 2$ and at the first cycle to $\mu_4 = \pm 4$ and $\mu_4 = \pm 6$ compare very well with the rotations estimated by the simple plastic analysis as discussed in Section 4.2.4. In the second cycles to $\mu_4 = \pm 4$ and $\mu_4 = \pm 6$, the rotational ductility demand decreased due to the shear deformations in the beam dominating the mode of deformation.

4.3.5 Beam Longitudinal Bar Strains and Forces

The strains in the longitudinal bars of the beam of Unit 2 were measured using manual DEMEC gauges in one side and clip gauges incorporating electrical resistance foil strain gauges on the other side. The gauge length was 204mm for both measuring devices. The strains in the reinforcement of both sides of the beam were of similar magnitude and therefore the average of east and west readings is presented in Fig. 4.18.

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![Fig. 4.15 - Components of Lateral Displacement of Unit 2 at Peaks of Load Runs.](image-url)
Fig. 4.16 Beam Curvature Ductility Factors of Unit 2.

\[ \phi_y = 0.0045 \text{ rads/m} \]

Fig. 4.17 - Beam Rotational Ductility Factors of Unit 2.

\[ \theta_y = 0.0036 \text{ rads} \]
The longitudinal bar strain profiles recorded at the peaks of the elastic cycles show clearly the tension shift effect near the ends of the beams. The strains tend to a zero tension shift near the midspan of the beam. The larger elastic strains recorded in the top bars reflect the influence of the different stiffness of the tension and compression columns during the test.

Yielding of the bars gradually spread in the beams from the column faces, as seen in Fig. 4.18. For comparison, Fig. 4.19 depicts the member average strains at the level of the reinforcing steel. It can be observed that the strains near the column face become erratic in the final loading runs because of the effect of the crack distribution in the beam, resulting in the member strain being overestimated in one gauge length and underestimated in the next. Fig. 4.20 shows the bar forces estimated from the measured strains using the procedure described in Section 4.2.6. The trends are very similar to those observed in Unit 1. In the cycles in the elastic range, load runs 5 and 6 in Fig. 4.20, the top reinforcement of the beam was subjected to larger forces than the bottom reinforcement. Plastic hinges regions extended a distance from the column faces equal to almost the effective depth of the beam, d. As in the beam of Unit 1, the tension shift effect rapidly decreased as the midspan of the beam was approached.

4.3.6 **Longitudinal Bar Stresses at Midspan**

Fig. 4.21 illustrates the level of stresses in the longitudinal bars in the region of the connection at the midspan of the beam. Some transfer of forces by the laps occurred between the bars in tension protruding from the precast concrete member and the "drop in" bars placed at the midspan of the beams. However, it appears that there was no such action between the "drop in" bars and the longitudinal bars at the other side of the beam.

4.3.7 **Stresses in the Transverse Reinforcement at Midspan**

The stresses in the vertical and horizontal legs of the stirrups surrounding the connection at midspan are shown in Figs. 4.22 and 4.23, respectively. These stresses were estimated from strains measured by double electrical resistance foil strain gauges opposite sides of the diameter of the stirrup. The stresses so calculated at the top and bottom vertical parts of the stirrups, shown in Fig. 4.22, were rather similar for the stirrups surrounding the overlapping bars but they were dissimilar for the stirrups located away from the connection region. The level of stresses in the stirrups in the connection region was well within the elastic range with the central stirrup consistently showing larger stresses.

Fig. 4.23 shows the stress distribution in the top horizontal portions of the stirrups. The higher values recorded in the stirrup outside the connection region were due to the transverse forces generated by the curtailment of the longitudinal bar near this section. The stresses in the connection region were very small and tended to increase in the final stages of the test.
Fig. 4.18 - Average Beam Longitudinal Bar Strains of Unit 2.
Fig. 4.19 - Member Strains at the Level of the Longitudinal Reinforcement of Unit 2.
Fig. 4.20 - Longitudinal Bar Forces in the Beam of Unit 2.
Fig. 4.21 - Stresses of the Beam Longitudinal Bars at the Connection at Midspan.
Fig. 4.22 - Stresses in the Vertical Transverse Reinforcement at Midspan of Unit 2.
Fig. 4.23 - Stresses in the Horizontal Transverse Reinforcement at Midspan - Top North Stirrups of Unit 2.
4.3.8 Elongation of the Beam

The total elongation of the beam of Unit 2 plotted against the lateral load can be seen in Fig. 4.24. The maximum elongation reached 32mm in load run 22 and in general it followed a similar pattern of the lengthening of the beam of Unit 1.

4.4 UNIT 3

4.4.1 General Behaviour

Unit 3 was connected at midspan by straight non-contact lap splices with two thirds of the longitudinal reinforcement in the beam lapped in this region (see Fig. 3.19). The lap length was equal to 23d_s and commenced at a distance of 1.27d from the column faces. Further details are presented in Section 3.2.2.1.

The test of this unit was carried out over a period of two weeks. The test was ended after completing two full cycles to \( \mu_a = \pm 8 \) at an interstorey drift of 2.7% when the lateral load capacity of the unit dropped to below 20% of the maximum recorded. The cumulative displacement ductility factor at failure was \( \Sigma \mu_a = 96 \) implying an available displacement ductility factor of \( \mu_s = 8 \). The overall hysteretic response of the unit was slightly better that the response of the previous two units, since the stiffness degradation was less pronounced and the energy dissipation was greater. It is believed that this improvement was caused by the additional transverse reinforcement provided at the beam ends, which was governed by the requirements to control bar buckling. Nevertheless, the amount of transverse steel provided did not prevent the large shear deformations in the plastic hinge regions of the beam becoming the principal mode of deformation in the final stages of the test.

Fig. 4.24 - Total Measured Beam Elongation of Unit 2.
Fig. 4.25 depicts the visible cracking of the beam of Unit 3 at different stages in the test. The cracking of the beam in the cycles in the elastic range extended to the cold joint between the precast members and the cast in place concrete at midspan. Several cracks became inclined and headed towards the compression region of the beam at the column face. Their width remained small with a maximum observed value of 0.2mm at the mid-depth of the beam. Additionally, some splitting cracks running parallel to the reinforcement being lapped formed in the top south side of the beam.

In the load runs 5 to 8 to $\mu_a = \pm 2$ the cracks in the beam at the column faces opened to between 2.5 and 3mm, suggesting that yield had commenced in the longitudinal bars in these regions and had probably penetrated inside the beam-column joints. These cracks remained open for the rest of the test. The diagonal cracks at the beam ends also opened but remained small. The maximum width measured at the mid-depth of the beam reached 0.4mm. New hairline fine cracks formed in the top north and bottom south sides of the beam in the spliced region.

The crack pattern observed in the beam in the cycles to $\mu_a = \pm 4$ showed the growth of the vertical cracks at the face of the column and the steeper diagonal cracks close to the beam ends (see Fig. 4.25 (a)). Some of these diagonal cracks became interconnected and the full depth open cracks resulted in some sliding shear displacements, especially in the south end where some spalling of the concrete cover also occurred.

The critical cracks in the plastic hinge regions of the beam were steeper than the cracks of the previous two units because of the larger amount of transverse reinforcement provided in these regions. However the number of stirrup-legs crossed by the critical cracks was approximately the same in all cases. Yielding of the stirrups in the plastic hinge regions of the beam probably also occurred in this test, since the observed vertical expansion of the north and south ends of the beam was 9 and 8mm in load run 17 and 12 and 23mm in load run 22, respectively.

Sliding shear concentrated in the south plastic hinge region in the final cycles of the test. The loss of the load capacity of 20% of the maximum lateral load recorded was caused by stiffness degradation. A sliding shear failure was observed when the test unit was pushed to an interstorey drift of 5%.

The crack pattern observed on the top and bottom surfaces of the beam at the midspan connection is shown in Fig. 4.26. Crack widths remained small on these surfaces and their inclination indicates that a diagonal compression field in the horizontal plane in the connecting region at midspan developed and formed a mechanism for transferring the forces between the lapped bars.

4.4.2 Load Displacement Response

The hysteresis loops in the loading cycles in the elastic range of the test of Unit 3 are shown in Fig. 4.27. The same behaviour as observed in Units 1 and 2 was typical of Unit 3. The second loop showed less energy being dissipated and was slightly softer. The measured "elastic" stiffness was only
Fig. 4.25 - Cracking of Beam of Unit 3 at Different Stages During Testing.
42% of that predicted in Section 3.2.1. The drift at the extrapolated first yield displacement was 0.34%, and the drift at the theoretical dependable lateral load capacity of the unit was 0.30%.

An early rounding of the loop in the first post-elastic loading cycle to $\mu_s = 2$, load run 5, occurred because of the presence of two layers of longitudinal tension steel in the beams where first yield took place. The lateral load capacity in this load run exceeded by 2.5% the predicted theoretical load $H_t$. Further post-elastic hysteresis loops are illustrated in Fig. 4.28. In load run 6 the lateral load attained was 5.1% below the theoretical load $H_t$. 
Fig. 4.27 - Lateral Load-Lateral Displacement Response of Unit 3 During Load Cycles in the Elastic Range.

Fig. 4.28 - Lateral Load-Lateral Displacement Response of Unit 3.
The post-elastic stiffnesses estimated as the slopes of the lines passing through the point at first yield and the point at maximum overstrength in load runs 13 and 14 were 3.3 and 1.7% of the initial stiffness, respectively. These are smaller values than those obtained in the previous two tests, probably because of the second layer of the reinforcement in the beam being subjected to less strain hardening. Using the elastic and average post-elastic stiffness as the basis for evaluating the energy dissipation of the ideal bi-linear loops, the first and second loading cycle to $\mu_a = \pm 2$ showed a normalized energy dissipation of 87 and 55%, respectively.

The response of Unit 3 in load runs 9 to 12 to $\mu_a = \pm 4$ showed a steady increase in lateral load capacity and very little strength degradation. Pinching of the loops was first noticed in load run 12. The normalized energy dissipated in these cycles was 66 and 45%, respectively.

In the four loading cycles to $\mu_a = \pm 6$, load runs 13 to 20, the hysteretic response was very pinched since large shear deformations dominated the response of the unit in these cycles. However, the stiffness did not degrade as in the previous units and the strength reduction was never more than 20% of the maximum lateral load capacity during the test. On the other hand, the energy dissipated during these cycles was rather similar to the other units, being 49, 33, 22 and 18% of the ideal bi-linear loops. The cumulative energy dissipation at the end of load run 20 amounted 38% of the ideal bi-linear
loops. The maximum measured strengths of 1.17Hₜ and 1.09Hₜ occurred in load runs 13 and 14 in the first loading cycle to μₜ = ±6, respectively. The maximum nominal shear stress in the beam plastic hinges was υ₀ = 0.28√Fₜ.

The first cycle to μₜ = ±8 in load runs 21 and 22 showed no reduction in the load carrying capacity. In fact, in load run 21 the lateral load attained was very similar to the load observed in load run 13 where the maximum lateral load strength was attained. In the second cycle the diagonal strut carrying the shear in the south end of the beam disintegrated and shortened and consequently a large degradation of stiffness was observed in load run 24. Further displacements to μₜ = -10 indicated that some reserve of strength was still available despite large sliding shear displacements concentrating at the south end of the beam.

4.4.3 Decomposition of Lateral Displacements

The components of the lateral displacement estimated following the procedure described in Section 3.9 are depicted in Fig. 4.29. The component caused by the fixed-end rotation of the beam ends accounted, on average, for 33% of the lateral displacement in the cycles in the elastic range and gradually decreased as the displacement ductility increased. The flexural response of the beam was also an important source of displacement in those cycles and in the cycles in the inelastic range to a new displacement ductility factor. In the second cycles in the inelastic range the flexural response decreased, being replaced by the displacements due to shear deformations in the plastic hinge regions. This was because the open cracks which formed after a new drift had been imposed increased the tendency of the plastic hinge regions to slide along the interconnected vertical cracks. Shear deformations became the dominant source of deformation from the load run 11 onwards. The columns and the beam-column joints made little further contribution to the deformations during the cycles in the inelastic range because they remained essentially elastic.

4.4.4 Beam Curvature and Rotational Ductility Factors

Shown in Figs. 4.30 and 4.31 are the beam curvature and rotational ductility factors. The beam curvature was measured over a gauge length of 170mm using the second set of linear potentiometers from the column faces, which were assumed to be unaffected by the strain penetration of the longitudinal bars inside the beam column joint region. As for previous two tests, the curvature and rotation at first yield were estimated from measured data. The curvature at first yield, ϕ₁, was estimated to be 0.0045 radians/m, a value larger than that predicted by a conventional moment-curvature analysis. It is believed that the main reason for this high initial curvature is the influence of the crack at the column face penetrating into the end regions of the beam as discussed in Section 4.3.4. The curvature ductility factors depicted in Fig. 4.30 exhibit a rather erratic distribution, but in general it can be seen that the curvature ductility demand decreases in the final stages of the test. This would have been because the large shear deformations contributed to a high percentage of the total displacement of the unit.
Fig. 4.30 - Beam Curvature Ductility Factors of Unit 3.

Fig. 4.31 - Beam Rotational Ductility Factors of Unit 3.
The computed rotation at first yield, \( \theta_y \), was 0.0038 radians. The rotational ductility factors in Fig. 4.31 show that the rotation demand decreased in the second cycles to the same ductility factor, because of the influence of the shear deformations in the plastic hinge regions of the beams. There is a very good agreement between the rotation estimated from the simple plastic analysis and the measured rotation in the cycles to \( \mu_s = \pm 2 \) and the first cycle to \( \mu_s = \pm 4 \). The rotational ductility factor demand was of the same order as the displacement ductility factor imposed on the unit, following the trends of the tests on the other two units.

4.4.5 Beam Longitudinal Bar Strains and Forces

The strain in the longitudinal bars at the beam of Unit 3 were measured using electrical resistance foil strain gauges. Unfortunately these measuring devices failed to provide data when the tensile strains in the reinforcement exceeded 1.0 to 1.5%. That is, in the first runs into the inelastic range the strain gauges near the faces of the column failed and as the test progressed, and the plastic hinges spread along the beams, the set of strain gauges in these regions debonded. Hence in this section only member strains are showed. Fig. 4.32 shows the gradual spreading of the plastic hinge regions and the residual tensile strains which accounted for the elongation of the beam.

Fig. 4.33 illustrates the bar forces in the outer layers of the longitudinal reinforcement of the beam, estimated from the strains measured before failure of the strain gauges. Unlike the two previous tests, the distribution of stresses in the top and bottom outer bars at the beam ends in the cycles in the elastic range was very similar, despite the lateral load carried by the compression column, being higher than the lateral load carried by the tension column. As in previous tests, yielding due to the spreading of the plastic hinge regions was confined to a distance from the column faces within the effective depth of the beam. In the top layer some yielding of the bars was also detected in the region of curtailment of the reinforcement. The tension shift effect of \( d \), suggested by the Concrete Design Code [NZS 3101 (1982)], formed a conservative envelope. Finally, large compression strains appear to have occurred in the reinforcement near the column faces. In this case the second layer of compression steel might not contribute much to carrying the compression force in the reinforcement because it is close to the neutral axis of the beam at the faces of the columns.

4.4.6 Longitudinal Bar Stresses at Midspan

The bar stresses in the connection region illustrated in Figs. 4.34 to 4.36 show that in most of the strain gauged bars yielding spread from the column faces to the commencement of the lap. The average bond stresses estimated in the intermediate section of the two splices of the reinforcement protruding from the side of the precast member subjected to tension were 5.6MPa in the outer layer of the top bars, 2.3MPa in the inner layer of the top bars and 2.7MPa in the outer layer of the bottom bars. It is of interest to observe that some lapping action actually occurred in this region. Bond forces in the bars in this region were transferred in the vertical plane required to balance the shear forces in the beams and in the horizontal plane to the lapping bars.
Fig. 4.32 - Member Strains at the Level of the Longitudinal Reinforcement of Unit 3.
Fig. 4.33 - Longitudinal Bar Forces in the Beam of Unit 3.
Fig. 4.34 - Longitudinal Bar Stresses in the Lapped Region - First Layer Top Bars of Unit 3.
Fig. 4.35 - Longitudinal Bar Stresses in the Lapped Region - Second Layer Top Bars of Unit 3.
Fig. 4.36 - Longitudinal Bar Stresses in the Lapped Region - First Layer Bottom Bars of Unit 3.
4.4.7 Elongation of the Beam

The total elongation of the beam, depicted in Fig. 4.37, shows almost the same trends as the test of Unit 2. A maximum total elongation of 35mm was recorded in load run 24.

4.5 CONCLUSIONS

1. Units 1, 2 and 3 were beam-column subassemblages with longitudinal reinforcement in the precast beams connected by various splice details in a cast in place joint at midspan. During the seismic load tests the connections behaved very satisfactorily and developed the capacity of the subassemblages. The commencement of the laps at 1.46d, 1.23d and 1.27d from the column faces of the three units did not have a detrimental effect on behaviour of the test units.

2. Observations made during this series of experiments showed the plastic hinge regions of these relatively short beams extended a distance from the column face which was approximately equal to the effective depth of the beam. It is then recommended that splices and other types of connections may commence at a distance as close as d to the column face.
3. The large shear deformations observed in these tests were the result of the small span/depth ratio of the beams and hence of the relatively high shear stress of $0.28\sqrt{f_c}$ to $0.37\sqrt{f_c}$ in the plastic hinge regions. These shear deformations affected the energy dissipation characteristics and stiffness, but the theoretical flexural strength of the units was maintained during the tests. The extra amount of transverse reinforcement in the plastic hinge region of Unit 3 resulted in only a slight improvement in performance compared with Units 1 and 2.

4. The estimated tensile forces in the beam longitudinal reinforcement indicate that the recommendations of the Concrete Design Code (1982) of using a tension shift equal to $d$ appear to be too conservative.

5. Significant beam elongations after yielding of the longitudinal reinforcement were observed in these tests. The main reason for beam elongation is the residual plastic tensile strain in the longitudinal reinforcement because in beams with equal amounts of top and bottom steel the steel in compression is subjected to a stress level below of that sustained by the bars in tension.

6. The stiffness of these Units was very low compared with the theoretical predictions using normal design office procedures. The main two reasons are that the curvature diagram does not follow the bending moment diagram and that strain penetration of the beam and column bars occurs in the beam-column joint region resulting in a fixed-end rotation.
CHAPTER 5
TEST RESULTS FROM UNIT 4

5.1 INTRODUCTION

This chapter contains test results from the H-shaped Unit 4, which comprised two precast concrete components with diagonally reinforced beam elements connected by bolts and steel plates at the midspan of the beam. The design of this unit was discussed in Section 3.2.2.2 and complete reinforcing details are illustrated in Fig. 3.6. The reinforcement lay-out for the beams involved strong end regions to relocate the plastic hinges away from the beam ends and to concentrate all inelastic deformations at midspan. This arrangement has occasionally been used in ductile frames of buildings in New Zealand because the Concrete Design Code [NZS 3101 (1982)] requires the use of diagonal reinforcement to carry part of the shear force in the regions expected to respond inelastically during a seismic event when the nominal shear stress at overstrength \( \nu = \frac{V_0}{bd} \) exceeds 0.30\( \sqrt{f_c} \). Another advantage of this arrangement is that the current Concrete Design Code allows a reduction in the amount of transverse steel required in beam-column joints when plastic hinges are designed to form in a region away from the column faces.

5.2 GENERAL BEHAVIOUR OF UNIT 4

The first test of Unit 4 was completed within one week. After undergoing the loading cycles up to two full cycles to \( \mu_a = \pm 4 \) the lateral load capacity of the unit dropped to 80% of the maximum recorded load. At this stage it was decided to terminate the test and to evaluate the possibility of retrofitting the damaged regions of the unit to enable it to be capable of a full ductile performance. A cumulative displacement ductility factor of \( \Sigma \mu_a = 24 \) had been imposed on the unit before ending the first test. The available displacement ductility factor obtained from the cumulative displacement ductility factor was \( \mu_a = 3 \) (see Section 3.7) that corresponds to a limited ductility performance.

Fig. 5.1 shows the beam of Unit 4 at two stages during the first test. Cracking in the beam was very symmetrical in the loading cycles in the elastic range. Most of the cracks appeared in the strong ends adjacent to the column. These cracks spread at regular intervals and their width did not exceed 0.3mm. Larger cracks, up to 0.6mm wide, were observed to propagate through the vertical construction joints between the precast concrete elements and the cast in place concrete in the midspan region. Only two hairline cracks, with a width of 0.1mm crossed the midspan connection.

In the first loading cycles in the inelastic range up to \( \mu_a = \pm 2 \), cracks concentrated around the bend of the diagonal bars. These cracks were up to 3mm wide. It was also observed that the concrete at the mid-depth of the beam near the steel straps had become loose and there were some
Fig. 5.1 - Cracking of the Beam of Unit 4 at Different Stages During the First Test.

(a) At $\mu_\Delta = -2x2$

(b) At $\mu_\Delta = -4x2$

Fig. 5.2 - Plan View of Top South End of the Beam of Unit 4 at the End of the First Test.
incipient signs of spalling of the concrete cover in this region. In further loading runs to $\mu_0 = \pm 2$, an out of plane horizontal movement (swelling) of 5mm was noticed in the concrete cover near the bends of the diagonal reinforcement, spalling of the concrete cover was also observed as it can be seen in Fig. 5.1 (a). The main cracks in the beam commenced in the concrete at the bends of the diagonal reinforcement. At the mid-depth of the beam they changed direction and, penetrated with an inclination of $35^\circ$ into the strong ends, heading towards the compression regions of the beam at the face of the columns. Surprisingly no major cracking occurred in the central region where the inelastic actions were expected to take place.

Extensive damage to the concrete in the strong ends of the beam occurred during the loading cycles to $\mu_0 = \pm 4$. At this stage large cracks at the top and bottom sides of the beam running between the D24 and D28 bars in the strong ends were also noticed. It is likely that these cracks had developed in the previous stages. Fig. 5.1 (b) shows the observed damage of the beam of Unit 4 at the end of the test. It is evident that the detailing of the reinforcement in the so called strong ends did not enforce the inelastic deformations to occur at the midspan of the beam as intended in the design, and therefore the simple truss model of Fig. 3.9 cannot be used to explain the behaviour of Unit 4.

Fig. 5.2 shows a close up of the top south end after removing the loose cover at the end of the test. A close inspection of the damaged region showed that the D28 bars in the strong ends had bent outwards as a result of bursting forces in the transverse direction and the lack of adequate reinforcement at the commencement of the anchorage hook (see Fig. 5.2). After removal of the concrete in the damaged regions it was also found that the concrete in the strong ends in contact with the D24 bars beside the 30x10mm steel straps had been completely crushed. There are two main reasons that explain the crushing of the concrete around the D24 bars. The first is associated with the effect of yield penetration of the D24 bars into the strong ends of the beam. The strains in these bars exceeded the yield strain and as a result the D24 bars elongated and slipped relative to the surrounding concrete, shearing it with its deformations. In the design of Unit 4 the current Concrete Design Code [NZS 3101 (1982)] recommendations regarding minimum bend radii for bars had been provided to reduce crushing of the surrounding concrete due to high local bearing stresses. However, these recommendations do not consider the shearing effect due to the local bar slip. The second reason is the high bearing stresses in the concrete around the D24 bars. Since in the test a large vertical crack appeared beside the 30x10mm steel straps, the shear carried by this reinforcement could not be transferred by a direct strut from the node at the bend of the diagonal reinforcement but rather by dowelling of the D24 bars inside the strong ends as depicted in Fig. 5.3. The bearing conditions around the outer diagonal bars were critical because they had to transfer not only the vertical component of the diagonal bar but also the vertical force of the straps. Crushing of the concrete surrounding the outer diagonal bars was seen to extend approximately 145mm or $6d_b$ from the vertical crack (see Fig. 5.3).

The average bearing stress, $f_b$, around one of the outer bars can be estimated as

$$f_b = \frac{\left(\frac{T_v}{4} + \frac{S_T}{2}\right)}{6d_b^2}$$

(5.1)
Fig. 5.3 - Transfer of Beam Shear in the Critical Region at the Bend of the D24 Diagonal Bars.

Fig. 5.4 - Free Bodies with Forces Acting upon Unit 4.

Fig. 5.5 - Refined Three Dimensional Truss Model of Unit 4.
where $T_v$ and $S_T$ are vertical forces in the diagonal bar and in the strap, respectively and $d_b$ is the
diameter of the bar as indicated in Fig. 5.3.

As a first approximation, it will be assumed that the truss mechanism of Fig. 3.9 developed
at the theoretical lateral load capacity $H_a = 636\text{kN}$. Thus, the bearing stresses around the diagonal bars
associated with this mechanism can be readily estimated. Relating the forces $T_v$ and $S_T$ in Eq. 5.1 to
$H_a$ and using the nominal bar diameter of the diagonal bars, $d_b = 24\text{mm}$, it can be found that $f_n = 58\text{MPa}$.

It is obvious that such a high bearing stress could not be sustained by the concrete in the
strong region of the beam where local crushing was detected, particularly since the concrete was not
confined.

A more realistic assessment of the likely bearing stresses acting during the test of Unit 4 can
be obtained from the free bodies shown in Fig. 5.4 in conjunction with the refined three dimensional
truss model illustrated in Fig. 5.5. The diagonal crack that penetrated into the strong regions of the
beam of Unit 4 is modelled as shown in Fig. 5.4. The free bodies in Fig. 5.4 imply that the shear in
the beam was transferred in this region not only by the vertical straps and the diagonal bars through a
direct strut but also by some of the stirrups in the beam ends.

The truss model in Fig. 5.5 is used to estimate the shear carried by the stirrups. It is
assumed, as it will be shown later, that the horizontal portions of the diagonal bars are equally stressed
along the strong ends and that the outer bars at the critical region have zero stress. The analysis yields
the following results:

Node J: No transverse force can be transferred in the horizontal plane since it is assumed that
splitting has already occurred and the outer bars have zero stress.

The force $T_v/4$ in this node does not change and therefore it does not require an
equilibrating force at the node. As a result no diagonal compression force in the vertical
plane can develop and hence the vertical stirrup at J is rendered ineffective.

Node K: A diagonal force $D_K$ can develop in this node. Its maximum value is limited by the yield
force $26.7\text{kN}$ of the stirrup.

$$D_K = \frac{26.7}{\sin 64.5^\circ} = 29.6\text{kN}$$

$$H_{KL} = D_K \frac{\cos 64.5^\circ}{\cos 32.8^\circ} = 15.2\text{kN}$$

$$F_K = H_{KL} \sin 32.8^\circ = 8.2\text{kN}$$
Note that force $H_{KL}$ can be transferred because of the horizontal tie at node L. The force $F_k$ is required to balance the transverse component of the diagonal force $H_{KL}$.

Node L: As in previous nodes the diagonal force $D_L$ is limited by the yield strength of the vertical stirrup at the horizontal tie. In addition the D28 bar is stressed to balance the longitudinal components of forces $D_L$ and $H_{KL}$.

$$D_L = \frac{26.7}{\sin 55.4^\circ} = 32.4 \text{kN}$$

$$F_L = H_{KL} \sin 32.8^\circ = 8.2 \text{kN} \ (0.31 f_y)$$

$$T_L = H_{KL} \cos 32.8^\circ + D_L \cos 55.4^\circ = 31.2 \text{kN} \ (0.16 f_y)$$

Node M: Only in-plane forces are necessary for equilibrium in this node. The diagonal force $D_M$ is limited by the yield strength of the stirrup.

$$D_M = \frac{26.7}{\sin 64.5^\circ} = 29.6 \text{kN}$$

$$T_M = T_L + \frac{D_M}{\cos 64.5^\circ} = 99.9 \text{kN} \ (0.52 f_y)$$

The vertical force carried by the 30x10mm straps can be estimated as the difference between the measured shear in the beam and the shear taken by the stirrups as shown in the above calculations. Table 5.1 illustrates this procedure and estimates the difference between the moment predicted with the distribution of forces derived from the analysis and the one obtained from the direct readings of the load cells of the test frame. The symbols in Table 5.1 are illustrated in Fig. 5.4.

The moment calculated using the estimated distribution of forces in the strong ends of Unit 4 gives an accurate value when compared with the measured value except in the semi-cycle of loading to $\mu_s = 2\times1$ where the straps were not yet activated because the compression force at midspan was mainly carried by the concrete. The difference can be assigned to the moment caused by the shear force carried through aggregate interlock ($V_{ag}$ in Fig. 5.4).

Table 5.2 shows the bearing stresses calculated from Eq. 5.1 using the results obtained in Table 5.1. These stresses are more realistic estimates of values which could have been attained during the test.

In the previous analysis it was assumed that splitting of the concrete between the D24 diagonal bars and the D28 outer bars in the strong ends of the beam had already occurred. An analysis to determine that this splitting was predictable is conducted in the following paragraphs.
Table 5.1
Bar Forces Acting on Unit 4

<table>
<thead>
<tr>
<th>$\mu_\Delta$</th>
<th>$V_{\text{ned}}$ (kN)</th>
<th>$M_{\text{ned}}$ (kNm)</th>
<th>$T_H$ (kN)</th>
<th>$T_v$ (kN)</th>
<th>$S_1$ (kN)</th>
<th>$S_2$ (kN)</th>
<th>$S_T$ (kN)</th>
<th>$M_{\text{calc}}$ (kNm)</th>
<th>$M_{\text{calc}} / M_{\text{ned}}$</th>
</tr>
</thead>
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<tr>
<td>2x1</td>
<td>522</td>
<td>549</td>
<td>515</td>
<td>267</td>
<td>107</td>
<td>53</td>
<td>-</td>
<td>475</td>
<td>0.87</td>
</tr>
<tr>
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<td>520</td>
<td>546</td>
<td>515</td>
<td>267</td>
<td>107</td>
<td>53</td>
<td>93</td>
<td>525</td>
<td>0.96</td>
</tr>
<tr>
<td>4x1</td>
<td>532</td>
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<td>515</td>
<td>267</td>
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<tr>
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<td>517</td>
<td>542</td>
<td>515</td>
<td>267</td>
<td>107</td>
<td>53</td>
<td>90</td>
<td>523</td>
<td>0.96</td>
</tr>
</tbody>
</table>

(1) In the beam at the column face

Table 5.2
Bearing Stresses in the Critical Region of Unit 4.

<table>
<thead>
<tr>
<th>$\mu_\Delta$</th>
<th>$T_v / 4 + S_T / 2$ (kN)</th>
<th>$f_s$ (kN)</th>
<th>$f_s / f'_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2x1</td>
<td>67</td>
<td>19</td>
<td>0.54</td>
</tr>
<tr>
<td>-2x1</td>
<td>113</td>
<td>33</td>
<td>0.94</td>
</tr>
<tr>
<td>4x1</td>
<td>119</td>
<td>35</td>
<td>1.00</td>
</tr>
<tr>
<td>-4x1</td>
<td>112</td>
<td>32</td>
<td>0.91</td>
</tr>
</tbody>
</table>

Fig. 5.6 shows the forces required for equilibrium at the node where the diagonal bars are bent and their distribution in the horizontal plane. It is assumed that the diagonal bars are equally stressed after and before the bend. For equilibrium in Fig. 5.6 (a)

$$R_H = \frac{T_v + S_T}{\tan \beta} + T_H - T$$  \hspace{1cm} (5.2)

One half of the force $R_H$ on the outer bars needs, in the absence of any transverse reinforcement, be transferred from the diagonal bars to each side of the beam by pure shear through the concrete (see Fig. 5.6 (b)). The average shear stress, $\tau_s$, associated with this mechanism is

$$\tau_s = \frac{R_H}{2A_t}$$  \hspace{1cm} (5.3 a)

or, substituting Eq. 5.2 in Eq. 5.3 (a)
Fig. 5.6 - Forces in the Bend Region of Diagonal Bars before Longitudinal Splitting of Concrete.
where $A_r$ is the effective friction area. The area $A_r$ is difficult to estimate since there is a complex distribution of stresses along the vertical plane between the outer and inner bars. In lieu of a better approach, it will be assumed that this area extends between the lines at the initiation of the bend of the second layer of bars, the concrete surface at the top and the centre line of the extension of the vertical hook of the first layer of bars as shown in Fig. 5.6 (c). That is:

$$A_r = 145 \times 214 = 31030 \text{mm}^2 \quad (5.4)$$

Now substituting $A_r = 31030 \text{mm}^2$, $\beta = 45^\circ$ and the values estimated from Table 5.1 for Unit 4 during $\mu_s = 2\times1$, $T_H = 515kN$, $T_v = 267kN$, $T = 580kN$ and $S_r = 93kN$ in Eq. 5.3 (b) gives

$$\tau_r = 4.8 \text{MPa} \quad (5.5)$$

To rely on the shear transfer mechanism the average shear stress shall be smaller than the diagonal tensile strength of concrete $f'_{ct}$, which can be assumed equal to the uniaxial tensile strength of the concrete, $f'_t$. The uniaxial and splitting strength of concrete are also related. An expression given by Collins and Mitchell (1991) is

$$f'_{ct} = 0.65f'_t \quad (5.6)$$

Substituting the measured splitting strength of the concrete of Unit 4, $f'_t = 3.9\text{MPa}$ (see Table 3.2) in Eq. 5.6 results in

$$f'_{ct} = 2.5\text{MPa}$$

which is a significantly smaller value than the estimated value of $\tau_r$. Therefore, the above analysis could have been used to predict a diagonal tension failure between the D24 diagonal bars and D28 outer bars as it was observed in the test of Unit 4.

Consequently, it can be concluded that the refined three dimensional truss model yields acceptable results. The model shows that the diagonal tension failure of the node at the bend of the diagonal bars was caused by the lay-out of longitudinal reinforcement used in the strong regions of the beam. These factors were not taken into account in the initial design.

The refined three dimensional truss model can also be used to explain the excellent behaviour of a diagonally reinforced unit with relocated plastic hinges tested by Bull (1978). In this unit the diagonal bars and the other bars provided for relocating the plastic hinge were in the same vertical plane and therefore no large bursting forces were expected.
5.3 **REPAIR OF UNIT 4**

Unit 4 was repaired after testing and retested for two purposes:

a) to verify the refined three dimensional truss model described above, and

b) to demonstrate that this type of connection can be designed to give an adequately ductile performance in moment resisting frames resisting earthquake forces.
To repair Unit 4 the damaged concrete was removed, the reinforcement detail appropriately modified and the damaged concrete replaced. Fig. 5.7 shows the reinforcement detail of Unit 4 after repair as it would be seen from a vertical plane passing between the diagonal bars and Fig. 5.8 shows a close up of the reinforcement in one of the repaired ends of the midspan region. The repaired unit is referred to as Unit 4r. Several measures were taken to alleviate the problem of crushing of the concrete surrounding the D24 diagonal bars, namely:

a) one of the 30x10mm steel straps was cut and anchored around the inner diagonal bars to distribute more evenly the bearing stresses between inner and outer bars,

b) transverse D24 rods were placed in contact with the diagonal bars to allow the diagonal bars to pivot in the rods avoiding shearing off the surrounding concrete,

c) higher strength concrete (60MPa) was cast in the repair space, and

d) 4-R16 ties were placed between the hooks of the outer D28 bars. This reinforcement was designed to provide the clamping force required to mobilize a shear friction mechanism to transfer the force \( R_u/2 \) (see Fig 5.6 (b)) at an overstrength of 1.25 in the diagonal bars. It was assumed that the entire beam shear would be transferred at point \( \text{aa'} \) in Fig. 5.6 (a) by the force \( T_v + S_r \). The hooks of the R16 ties were tack welded to avoid opening.

5.4 GENERAL BEHAVIOUR OF UNIT 4r

Unit 4r was tested 19 weeks after the completion of the test of Unit 4. The test sequence applied to this unit was identical to that of the original subassemblage. The test took a week to conduct and was terminated when the lateral load capacity at the fourth cycle to \( \mu = +6 \) dropped to 72% of the maximum measured load. The cumulative displacement ductility imposed to the third cycle to \( \mu = -6 \) was \( \Sigma \mu = 60 \), which corresponds to an available displacement ductility factor of \( \mu_a = 7.5 \).

The condition of the beam of Unit 4r at various stages is illustrated in Fig. 5.9. Cracking in the beam in the loading cycles in the elastic range was mainly confined to the strong ends with only one diagonal crack crossing the midspan region. The cracks in the critical region at the ends of the strong region followed the construction joints between the cast in place concrete at midspan and the high strength concrete used in the repaired area. In the top and bottom surfaces of the beam, cracks between the outer D28 bars and the inner D24 bars connections at the bend of the D28 bars were observed. These cracks were of the same shape as the main splitting cracks that penetrated the strong ends in Unit 4 (see Fig. 5.2). The maximum crack width of 0.5mm recorded in these loading cycles was observed in the diagonal cracks at the mid-depth of the beam close to the column faces. All cracks which opened for one direction of loading practically closed when the loading direction was reversed.
Fig. 5.9 - Cracking of the Beam of Unit 4r at Different Stages During the Test.
Cracking in the beam concentrated in the critical region in the cycles to \( \mu_\lambda = 2 \), where crack widths reached 2.6mm in the extreme tension fibre. Crack widths elsewhere remained very small including those crossing the central region of the beam.

In the loading cycles to \( \mu_\lambda = 4 \) the main cracks in the beam spread from the critical end region towards midspan. These cracks were large with crack widths up to 8mm. It is evident that most of the inelastic deformations had been confined to within the central region of the beam (see Fig. 5.9 (a)). Nevertheless, the cracks in the beam at the column faces grew to 3mm in width in the extreme tension fibres indicating that yield of the beam longitudinal bars had penetrated into the beam-column joints.

In the final cycles to \( \mu_\lambda = 6 \), wide diagonal cracks developed in the strong ends of the beam, especially in the north end (see Fig. 5.9 (b) and (c)). Associated with this crack pattern was a large sliding shear movement and grinding of the concrete between cracks. It is very likely that the stirrups were subjected to very large strains beyond the elastic range, since a vertical growth of 18mm in the beam depth was recorded in the north end. The repaired Unit displayed a sliding shear failure as a result. No fracture of the reinforcement at the bends of the diagonal bars due to strain age embrittlement occurred during the two tests. However, these results have to be carefully interpreted since the tests were conducted at a temperature of 17°C and embrittlement is very dependent on the temperature conditions.

5.5 LOAD-DISPLACEMENT RESPONSE

The lateral load versus lateral displacement response of the original and repaired unit was significantly different, as can be seen in Fig. 5.10.

The initial elastic stiffness of Unit 4 calculated as the average stiffness at \( \pm 0.75H_i \) was 54% of the theoretical stiffness presented in Section 3.2.1. This apparently more flexible behaviour was because the effect of the tension shift on the curvature distribution along the beam, and of the deformations in the beam-column joint region which, amongst others, were not accounted for in the initial theoretical analysis.

In the loading cycles in the inelastic range Unit 4 never attained the theoretical load predicted but the values measured in the first loading cycle were very close to it. Therefore a post-elastic stiffness of 0 was chosen for the ideal bi-linear response in order to normalize the energy absorbed in the measured hysteretic response.

The lateral load measured in load runs 5 and 6 to \( \mu_\lambda = \pm 2 \) was 98% and 97% of \( H_\lambda \) where \( H_\lambda \) is the theoretical load capacity calculated using the simple truss analysis assuming that the diagonal reinforcement in the beam yields and the reinforcement elsewhere remains in the elastic range. The second load cycles to the same ductility level showed some decay in load carrying capacity as well as some pinching in the measured response. The normalised energy absorbed by the loops in the first and second cycle was 80% and 43%, respectively.
Fig. 5.10 - Lateral Load-Lateral Displacement Response of Units 4 and 4r.
In load runs 9 and 10 to $\mu_a = \pm 4$ the lateral load attained was $0.99H_e$ and $0.97H_e$, respectively, but the capacity of the unit dropped significantly in the second cycle to the same ductility level. In load run 12 the lateral load measured was 80% of that attained in load run 10. The normalised energy absorbed by the loops in the first and second cycle to $\mu_a = \pm 4$ was 58% and 43%, respectively.

For the repaired unit, Unit 4r, the measured initial stiffness was 79% of the measured stiffness in the elastic range of the original unit. This reduction was caused by the increase in flexibility of the beam-column joints as well as those cracked regions in the beam which were not treated during the retrofitting of the unit. It is likely that, after several loading cycles in the elastic range in the first test, the stiffness of the columns had also diminished.

The measured post-elastic stiffnesses of the positive and negative loading cycles of Unit 4r were rather large because of the significant overstrengths attained. The calculated values were 10.8% and 9.9% of the initial elastic stiffness.

The theoretical lateral capacity $H_e$ was exceeded by 23% and 20% in the first loading cycle in the inelastic range to $\mu_a = \pm 2$. The second loading cycle to the same ductility factor also exceeded the load $H_e$ but there was a slight drop in load carrying capacity. The normalised energy absorbed in these cycles was 81% and 39%, respectively.

The load runs 9 and 10 to $\mu_a = \pm 4$ the unit showed a steady increase in lateral load capacity, reaching overstrengths of $1.37H_e$ and $1.35H_e$. The lateral load attained at the peak of the second loading cycle was slightly smaller than that of the first loading cycle. The normalised energy absorbed in these two cycles was 77% and 63%, respectively. The consequence of reaching such a high strength in the diagonally reinforced midspan region was that the moment capacity of the beams at the column ends was equalled and therefore yielding of the steel commenced at the ends. Yielding of the steel in the strong ends had been discouraged by adding sufficient longitudinal steel there so as to comply with a capacity design philosophy of using a nominal overstrength factor of 1.25 in the diagonally reinforced region.

In the load runs to $\mu_a = \pm 6$, plastic hinges had formed in the beams at the face of the columns. The maximum lateral load of $1.41H_e$ and $1.37H_e$ was measured in load runs 13 and 14 respectively. The large cracks in the strong ends resulted in some loss of aggregate interlock as a manner of transferring the high reversing shear forces of $V/\beta_d = 0.45V_{c, e}$ based on a concrete compressive strength of $f'_c = 60$MPa. Consequently, there was a gradual degradation of the concrete in one of the strong ends of the beam and with it a loss of stiffness and capacity. In load run 19 the recorded lateral load attained reached 72% of the maximum load measured in load run 13. Further displacements showed that the lateral load capacity had been exhausted, as is seen in Fig. 5.10. A measure of the deterioration of the unit is given by the index of the normalized energy which was 71%
of the ideal bi-linear loops in the first load cycle to \( \mu_a = 6 \) and 54\% and 40\% in the subsequent two cycles, respectively. The cumulative energy absorbed including the third cycle to \( \mu_a = 6 \) was 58\%.

It is believed that this high overstrength was caused by a combination of strain hardening following the previous test and by strain ageing of the reinforcement. It is of particular importance to recognise that for diagonally reinforced members an overstrength value of 1.25 might be unconservative. This is because, for the strain history expected in a diagonal reinforcing bar, the concept of the shifted envelop to describe the stress-strain behaviour of New Zealand manufactured steel is not necessarily appropriate (see Section 2.5.2.5). This assumption was used by Andriano and Park (1987) to estimate the overstrength factors of beams using New Zealand steel. Regarding strain ageing it appears that this effect adds an additional overstrength, as indicated by the limited testing already discussed in Chapter 2.

5.6 DECOMPOSITION OF LATERAL DISPLACEMENTS

Fig. 5.11 illustrates the decomposition of the lateral displacements imposed at the peak of each load cycle for Unit 4. The main source of displacement in the loading cycles in the elastic range were the flexural deformations in the strong ends of the beam. These deformations suggest that strong ends in skeletal structures of this type should not be modeled by rigid end blocks. Shear deformations at midspan became gradually more important with the progression of the test. These were caused by crushing of the concrete at the bend of the diagonal reinforcement and were concentrated over a very small length in the beam span, more like sliding shear action. However, flexural deformations in the strong ends contributed a significant percentage of the lateral displacement throughout the test.

Flexural deformations in the strong ends were also the main source of displacement of Unit 4r in the initial loading cycles in the elastic range as illustrated in Fig. 5.12. Shear deformations occurring in the beam and in the beam-column joint panel were much higher than those occurring in the previous test, as a result of the cracked conditions at the beginning of the second test and the effect of the previous loading history. Flexural deformations in the central region of the beam increased in the intermediate stages of the test, owing to the inelastic deformations taking place in this region. However, these deformations became less important in the cycles near the end of the test because of the migration of the plastic hinges towards the beam ends which resulted in large shear and flexural deformations in the strong end regions. Shear deformations, mainly arising from the north end of the beam, governed the displacement response of the unit in the final cycle prior to the end of the test.

5.7 CURVATURE PROFILES ALONG THE BEAM

The curvature distributions along the beams of Units 4 and 4r, obtained from linear potentiometer readings, are shown in Figs. 5.13 and 5.14. These plots are presented in terms of
Fig. 5.11 - Decomposition of Lateral Displacements of Unit 4.

Fig. 5.12 - Decomposition of Lateral Displacements of Unit 4r.
Fig. 5.13 - Curvature Distribution for the Beam of Unit 4.
Fig. 5.14 - Curvature Distribution for the Beam of Unit 4r.
curvature rather than curvature ductility factors. This was because an experimentally defined yield curvature was discarded, since it was felt that the readings obtained by the linear potentiometers placed in the critical region at the bend of the diagonal bars would have been largely affected by the strain penetration of the bars inside the strong ends, which led to a fixed-end rotation.

Two distinct trends can be observed in Figs. 5.13 and 5.14. In Unit 4 the large curvatures were concentrated in the critical regions and although penetrating into the strong ends they decreased near the column faces. On the other hand, in Unit 4r large curvatures also concentrated in the critical regions but after the flexural capacity of the strong ends was reached they also spread from the column faces towards midspan. Unlike the curvature distribution in Unit 4, the curvature in the central region of the strong ends of the beam of Unit 4r was very small since yielding of the longitudinal reinforcement in this zone was delayed.

The curvatures in the reversal load runs in Unit 4 were significantly smaller than in the initial or positive load runs, because of the shearing of the beam at the bend of the diagonal bars. The curvatures attained in both tests were moderate and certainly much smaller than expected the response was dominated by flexure in the central region of the beam. Note that the flexural response in the central region was very small in the final cycles of each test (see Figs. 5.11 and 5.12).

5.8 BEAM BAR STRAINS AND STRESSES

5.8.1 Strain Profiles of the Beam Diagonal Bars

The bar strains recorded by the 5mm electrical foil strain gauges on the D24 diagonal bars in Unit 4 are shown plotted in Fig. 5.15. The strain profiles in the loading cycles in the elastic range were very uniform and only decreased in the beam-column joints. The average tensile strain in the D24 bars at the face of the column was 22% larger than that estimated using conventional elastic theory analysis, which assumed a parabolic distribution of stresses for concrete and the measured properties for steel. One reason for this difference can be attributed to the outer D28 bars not fully contributing to the flexural capacity of the unit because of the poor anchorage conditions in the critical region at the bend of the diagonal bars.

Unfortunately many readings in the region where large inelastic strains were taking place became unusable due to early debonding of the strain gauges. Therefore only reliable readings are plotted in Fig. 5.15. It is obvious that yielding of the longitudinal reinforcement was not confined to within the midspan region but also spread inside the strong ends, reaching the column faces in the load cycles to $\mu_a = \pm 4$. 
During the repair of Unit 4 several debonded strain gauges were removed from the bars and were replaced by new ones. This permitted the complete strain profile of the longitudinal bars to be recorded, at least in the initial stages of the test on Unit 4r. The recorded strains of the D24 bars are plotted in Fig. 5.16 for the loading runs in the elastic and inelastic ranges for Unit 4r. In the stages in the elastic range, load runs 1 and 2, the strain profiles in the beam span showed a maximum strain at the column faces. The strain in the bars decreased slightly along the strong ends and peaked again in the central region of the beam. The measured strains at the face of the columns were, on average, 13% smaller than the strains measured in the same load runs in the test of Unit 4 and 6% larger than the theoretical strains predicted from the elastic analysis. Although it is not obvious that this difference was due to the effectiveness of the anchorage conditions of the D28 bars, that anchorage is certainly one of the reasons. As Fig. 5.17 illustrates the D28 bars were moderately stressed at the beginning of the anchorage in these load runs. Another reason could be the influence of the high strength concrete in reducing the lever arm for a given flexural demand. The effects of the high strength concrete are very difficult to quantify since the concrete near the face of the columns was composed of the two different concrete mixes used in the precasting and the repair.

5.8.2 Bar Stresses in the Reinforcement at the Bend of the Diagonal bars

Fig. 5.17 shows the stress level of the reinforcement at the bend of the diagonal bars estimated from measured strains in the test of the repaired unit, Unit 4r. Strain gauges were attached to the reinforcement in this regions in Unit 4r to monitor the transfer of forces in the repaired critical region of the beam.

As shown in Fig. 5.17 (a) the D28 bars were actively working since the beginning of the test. This indicates that it was possible to transfer forces between the diagonal bars and the bars located on the outer planes on the strong ends of the beams. Yielding of these bars finally occurred in the second cycle to $\mu_a = 4$ due to the high lateral load attained during the test at this stage. When Fig. 5.17 (a) is analyzed in combination with Fig. 5.17 (b) it is evident that the transverse ties provided an effective clamping force as required by a shear friction mechanism. Only the stresses estimated from measured strains in the horizontal tie on the bottom side of the bend of the north diagonal bars show an erratic behaviour. It is likely that bending of the ties affected the stress measurements, since only one strain gauge was attached to the side of the reinforcing bars.

The stresses in the vertical steel straps calculated from measure strains indicate that, as expected in the intermediate and final stages of the test of Unit 4r, the shear force carried by this reinforcement was close to half of the beam shear force (see Fig. 5.17 (c)). In the cycle in the inelastic range to $\mu_a = 2$, 34% of the shear was carried by the steel straps. This low value can be explained because the bars in the compression side have not yielded in tension and therefore they do not mobilize the vertical straps at this stage. In the loading runs to $\mu_a = 4x1$ and $\mu_a = 6x1$, the vertical steel straps carried 41% of the beam shear.
Fig. 5.15 - Strain Profiles for the Longitudinal Beam Bars of Unit 4.
Fig. 5.16 - Strain Profiles for the Longitudinal Beam Bars of Unit 4r.
Fig. 5.17 - Bar Stresses in Reinforcement at the Critical Region of Unit 4r.
Fig. 5.18 - Measured Beam Elongation versus Storey Shear.
5.9 BEAM ELONGATION

The measured beam elongation of Units 4 and 4r is plotted in Fig. 5.18 against the storey shear. It is evident that the elongation recorded in the test is significant. However, an even larger elongation would be expected if most of the rotation in the beams had been concentrated at the critical regions. The measured elongation was not so large because in the final stages of the test the main source of rotation in the beam shifted towards the beam ends and also because shear deformations became dominant at this stage.

5.10 CONCLUSIONS

1. The experimental results obtained from the simulated seismic load conducted tests on a diagonally reinforced precast concrete unit, with relocated plastic hinges connected at the midspan at the beam by bolted steel plates, indicated that careful detailing of the regions at the bend of the diagonal bars is required if a full ductile response is to be attained. Two main points need to be considered in the detailing of these regions. First, transverse forces may arise as a result of the lay-out of the longitudinal reinforcement and adequate transverse reinforcement needs to be provided for those forces. Second, a bearing failure caused by crushing and shearing of the concrete in contact with the diagonal bars may occur. In the tests on the repaired unit transverse rods in contact with the inside of the bend of the diagonal bars was shown to be an excellent method for preventing bearing failure. A refined three dimensional truss model can be used to explain the behaviour of the unit in the first test and to design the reinforcement for the repaired unit.

2. Strain age embrittlement of the reinforcing steel at the bends of the diagonal bars was not observed during the test and retest of the unit. Three quarters of the bars had been artificially strain aged to observe this behaviour. However, it cannot be concluded that this phenomena might not be critical since strain age embrittlement is very susceptible to low temperature values and the temperature during the tests was about 17°C.

3. An additional overstrength factor owing to strain hardening and strain ageing of steel should be considered when assessing the post-earthquake capacity of a diagonally reinforced concrete member with relocated plastic hinges. Higher shear forces than normally estimated are expected to develop in a beam of this type, in which the diagonal reinforcement has previously yielded into the strain hardening region. As a result plastic hinges may form in the strong ends of the beam or elsewhere in the structure.

4. The design of the midspan bolted connection using standard procedures for structural steel resulted in satisfactory performance. Several loading cycles where the welded reinforcement
yielded in tension and compression at the connection region were imposed in the tests and no distress was observed in this region.

5. In the initial loading cycles in the elastic range, up to 38% of the imposed lateral deflection was caused by the flexibility of the strong end regions of the beam. It is therefore recommended that allowances for this flexibility be made in structural analysis.